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PRESHOT MATERIAL PROPERTY INVESTIGATION FOR THE MIXED COMPANY SITE: SUMMARY OF SUBSURFACE EXPLORATION AND LABORATORY TEST RESULTS

John Q. Ehrgott

Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

October 1973

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This report summarizes the investigations conducted under Project LN 310, "Soil Sampling and Laboratory Testing for Constitutive Relations," in support of Mixed Company Event III, a 500-ton, high-explosive experiment conducted near Grand Junction, Colorado. The primary purpose of Project LN 310 was to provide a representative geologic profile of the Mixed Company site along with associated constitutive properties for use in the preshot two-dimensional ground shock calculations planned under Project LN 312. This report describes results from (1) a field investigation program consisting of a geologic survey, a refraction seismic survey, and an exploration boring and sampling program; (2) a laboratory test program consisting of static and dynamic uniaxial strain tests, isotropic compression tests, triaxial shear tests, and static tension tests; and (3) the analyses applied to the data obtained from both of these programs in order to develop a recommended site profile and matching set of constitutive properties in time to support preshot calculations. Typical test data are presented to illustrate general trends and/or response characteristics.

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MISCELLANEOUS PAPER S-73-62

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by

J. Q. Ehrgott

October 1973

Sponsored by Defense Nuclear Agency
Subtask SB209, Work Unit 11, "Laboratory Studies of the Response of Soil and Rock to Blast-Type Loadings"

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

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ABSTRACT

This report summarizes the investigations conducted under Project LN 310, "Soil Sampling and Laboratory Testing for Constitutive Relations," in support of Mixed Company Event III, a 500-ton, high-explosive experiment conducted near Grand Junction, Colorado. The primary purpose of Project LN 310 was to provide a representative geologic profile of the Mixed Company site along with associated constitutive properties for use in the preshot two-dimensional ground shock calculations planned under Project LN 312. This report describes results from (1) a field investigation program consisting of a geologic survey, a refraction seismic survey, and an exploration boring and sampling program; (2) a laboratory test program consisting of static and dynamic uniaxial strain tests, isotropic compression tests, triaxial shear tests, and static tension tests; and (3) the analyses applied to the data obtained from both of these programs in order to develop a recommended site profile and matching set of constitutive properties in time to support preshot calculations. Typical test data are presented to illustrate general trends and/or response characteristics.

PREFACE

This report is essentially a paper prepared for presentation at the Mixed Company/Middle Gust Project Review Meeting held at the DOD Nuclear Information and Analysis Center (DASIAC), Santa Barbara, California, 13-15 March 1973. It summarizes results of investigations conducted by the Soils and Pavements Laboratory (S&PL) of the U. S. Army Engineer Waterways Experiment Station (WES) during the period August 1971 through July 1972 under Mixed Company Project LN 310, "Soil Sampling and Laboratory Testing for Constitutive Relations." The project was sponsored by the Defense Nuclear Agency (DNA) under NWE Subtask SB209, "Propagation of Ground Shock Through Earth Media." Mr. C. B. McFarland was the DNA Project Officer for Subtask SB209. Supporting studies were conducted by Terra Tek, Inc., (TT) and Stanford Research Institute (SRI) under separate DNA funding.

The LN 310 Project Officer was Mr. J. Q. Ehrgott of the Soil Dynamics Division, S&PL, WES, who prepared and presented this paper. Drs. J. S. Zelasko and P. F. Hadala, Messrs. J. T. Gatz and R. E. Leach, and PVT J. R. Benham of the S&PL, WES; Messrs. R. L. Stowe, B. R. Sullivan, and R. A. Bendinelli of the Concrete Laboratory, WES; Mr. S. J. Green of TT; and Dr. C. F. Peterson of SRI made substantial constibutions to the project. The work was conducted under the general direction of Dr. J. G. Jackson, Jr., Chief of the Soil Dynamics Division, S&PL. Mr. J. P. Sale was Chief of S&PL during the conduct of the study.

BG E. D. Peixotto, CE, and COL G. H. Hilt, CE, were Directors of the WES during the investigation and the publication of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
tons (2,000 pounds)	907.185	kilograms
pounds (force) per square inch	0.6894757	newtons per square centimeter
kips (force) per square inch	0.6894757	kilonewtons per square centimeter
pounds (mass) per cubic foot	16.0185	kilograms per cubic meter
feet per second	0.3048	meters per second

CHAPTER 1

INTRODUCTION

Mixed Company Project LN 310, "Soil Sampling and Laboratory Testing for Constitutive Relations," was performed by the U. S. Army Engineer Waterways Experiment Station (WES) for the Defense Nuclear Agency (DNA) under NWE Subtask SB209. Terra Tek, Inc. (TT), and Stanford Research Institute (SRI) were also funded by DNA to support the project.

The Mixed Company program consisted of several high-explosive field tests, i.e., two 20-ton[1] tests (Events I and II), and a 500-ton test (Event III). The primary objective of Project LN 310 was to provide a representative geologic profile for the Mixed Company Event III site location with associated constitutive properties in a form suitable for use in two-dimensional (2D) ground shock calculations. Constitutive property recommendations were disseminated in June 1972 and were used as a basis for the mathematical model formulations in the preshot groundshock calculations performed under Project LN 312, "Theoretical Ground Motion Calculations." In addition, a grout mixture whose properties approximately matched the constitutive properties of the underlying sandstone at this site was developed for use in the placement of instrumentation for the 500-ton event.

The purpose of this report is to summarize the results obtained from a field investigation program consisting of a geologic survey, a refraction seismic survey, and an exploratory boring and sampling program (Chapter 2); a laboratory test program consisting of static and dynamic uniaxial strain tests, isotropic compression tests, triaxial shear tests, and static tension tests (Chapter 3); and the analyses applied to the data obtained from both of those programs in order to develop a recommended site profile and matching set of constitutive properties (Chapter 4).

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[1] A table of factors for converting British units of measurement to metric units is presented on page 8.

CHAPTER 2

FIELD INVESTIGATION

The geologic criterion for choosing the site of the Mixed Company tests was that the near-surface site profile should consist of a shallow stratum of sandy soil overlying a deep deposit of fairly homogeneous sandstone. During the summer of 1971, a preliminary geologic field survey and boring program was conducted at several possible site locations in Utah and in the Grand Junction, Colorado, area. Based on the information obtained from those studies, a site in Mesa County, Colorado, about 17 miles west-southwest of Grand Junction was selected. During the late summer and fall of 1971, the main field investigation was conducted at this site and consisted of an extensive surface refraction seismic survey, approximately 15 subsurface exploration borings, the opening of three test pits to expose the soil-rock interface, and mapping of the soil-rock interface across the site area through use of power auger borings. In February 1972, additional subsurface borings were drilled to obtain in situ water content and density measurements. Geologic inspections of the site were made during each investigation, in addition to laboratory classification tests and literature studies.

2.1 SITE GEOLOGY

The Mixed Company site, located in the northeastern part of the canyon lands section of the Colorado Plateau physiographic province, is on a broad plain on the northern flank of the Uncompahgre Plateau. The plain dips 1 to 2 percent to the west and northwest in the area of the site. Severe to moderate erosion has exposed cliff faces in the immediate vicinity of the site. A mesa approximately 1 mile north of the site rises about 150 to 200 feet above the plain. A 100-foot-deep valley or canyon with vertical walls lies approximately 1/2-mile to the west. About 1 mile south of the site, the terrain falls abruptly 100 to 200 feet into another canyon (Trail Canyon). The formations that underlie the site are exposed in both canyons.

Small intermittent streams cross a portion of the site, but no

continuous stream occurs within 2 miles of the site. The region drains into the Colorado River, which is located about 10 miles to the north and forms an arc around the northwestern end of the Uncompahgre Plateau. Most of the water used in the area is produced from deep wells that penetrate one or two water-bearing sandstone formations, i.e., the Entrada or the Wingate.

2.1.1 Structure. No evidence which would indicate the existence of major geologic faulting within the immediate site area was found. The closest mapped fault is the east-west trending Glade Park fault that appears to end about 4 miles east of the site. Several indications of minor faults appear in a mesa north of the Mixed Company site, but these are not believed to extend into the test area. South of the site, Mesozoic strata generally strike east-west. These strata dip less than 5 degrees to the north, i.e., away from the Uncompahgre Uplift. The existence of intermittent streams flowing north and west across the broad plain suggests that this trend also exists at the test site.

2.1.2 Stratigraphy. A stratigraphy column of the site area is shown in Figure 2.1. The entire site is covered by a relatively thin (i.e., 1- to 8-foot-thick) deposit of sandy clayey silt. The soil appears to become slightly cemented with depth. Underlying the soil cover is the 70-foot-thick Kayenta Formation of the Glen Canyon Group and Triassic period. The uppermost portion consists of weathered siltstone (approximately 4 feet thick), resulting in a somewhat indistinguishable interface between it and the overlying soil cover. The Kayenta is a fluvial deposit (water lain) which consists of lenticular to irregularly bedded layers of fine- to medium-grained sandstone, siltstone, and conglomerate with occasional layers or lenses of shale. Thus, there is a significant variation in the visual appearance of the samples from this formation although there were general correlations of materials between borings.

The Wingate Formation of the Glen Canyon Group lies beneath the Kayenta; it is encountered at a depth of approximately 70 feet and is believed to extend to about the 400-foot depth. It is a massive, fine-grained, crossbedded sandstone whose origin is predominantly eolian

(windblown). Because of its high porosity, it is one of the two water-bearing rocks found in the area.

Below the Wingate is the Chinle Formation, which is also of Triassic period. It was deposited on a widespread, low-lying alluvial plain with many lakes and consists mostly of red siltstone with thin, hard beds of limestone, conglomerate, and greenish siltstone. It is believed to extend to approximately the 500-foot depth where it rests unconformably on a very smooth erosional surface of the Precambrian basement rock. The Precambrian rock consists primarily of schist and gneiss with younger granitic intrusions.

The mesa north of the site consists of Jurassic rocks of the eolian Entrada Formation sandstone with a thin top cover of shale of the Summersville-Morrison Formations. These formations have generally been eroded away from the test site area; however, it is possible that local surface pockets of Entrada sandstone may still exist at the site. The Entrada sandstone is similar to the Wingate sandstone and is also a water-bearing rock.

2.2 REFRACTION SEISMIC SURVEY

A surface refraction seismic survey was conducted at the site in September 1971. The survey followed the three main subsurface boring lines which were positioned 120 degrees apart and which radiated out from the then-proposed 500-ton ground zero (GZ) location. Most of the seismic investigations were conducted using a hammer source with 150-foot lines and 15-foot geophone spacings. One 570-foot line was run using an explosive source to provide extended depth coverage.

The results indicated a low-velocity zone (2,000 to 3,000 fps) overlying a high-velocity zone (6,000 to 8,800 fps). A schematic fence diagram along the three boring lines is presented in Figure 2.2 to illustrate the subsurface profile as defined by the seismic survey. It should be noted that the final GZ location of Event III was at the location of Boring C10, as indicated in the diagram. The thickness of the low-velocity zone ranged from 9 to 15 feet. The results of the one deep survey indicated an additional higher velocity zone (14,000 fps)

beginning at a depth of approximately 90 to 130 feet. The deep survey was conducted in line with Borings C5 to C6, but extended farther east (see Figure 2.2).

2.3 SUBSURFACE EXPLORATION

The subsurface exploration investigation was conducted to (1) identify the subsurface material within a 500-foot radius of GZ, (2) obtain sufficient samples of each type material encountered for the laboratory test program, (3) obtain measurements of in situ water content and density in the soil and rock materials, (4) inspect the soil-rock interface for jointing, and (5) map the depth to rock within the general area. The main field investigation conducted in the fall of 1971 accomplished those objectives with one exception, i.e., questionable data were obtained regarding the in situ water content of the rock material. The NX-size core samples were allowed to air-dry after sampling and the 5-inch-diameter Shelby tube samples, drilled with water, were sealed after sampling, thereby possibly trapping drilling water in the material. Therefore, an additional set of borings was drilled in February 1972 using both air and colored water as drilling coolants. Samples from these borings were sealed immediately after recovery. These samples also supplemented the supply for the laboratory test program.

A plan view of the site showing all boring locations, the final location of GZ for Event III, and the location of the WES main gage line is given in Figure 2.3. The holes designated "C" and "U" were drilled in September 1971; the holes designated "NXA" and "UA" were drilled in February 1972.

Power auger borings were made in a 50- by 50-foot grid pattern over the site in an attempt to determine the depth of the soil overburden. The depth of auger refusal, however, indicated a highly variable depth to the soil-rock interface; i.e., refusal ranged from 1 to 15 feet. In truth, there is no uniform soil-rock interface at the site due to the variable degree of weathering of the near-surface Kayenta siltstone. Results from the main subsurface borings and three test pits indicated

a somewhat narrower variation in interface depth; i.e., a range from 1 to only 8 feet.

The variation of the composition properties of the soil zone with depth is shown in Figure 2.4. The average water content was between 5 and 6 percent. The wet density increased from 95 pcf at the surface to 120 pcf at the 3- to 4-foot depth.

The variation of composition properties with depth determined for the underlying rock is presented in Figure 2.5. The suspect water content data are not shown. Below the soil zone and above the 40-foot depth, a distinct horizontally layered profile is not evident in the composition property data. Since the questionable water content information was eliminated, the scatter in the composition properties is, most probably, due to the great variation in lithology (i.e., sandstones, siltstones, mudstones, and conglomerates) encountered in the Kayenta Formation.

Figure 2.6 is a schematic fence diagram along the boring lines showing the overburden soil thickness as defined from the borings. Superimposed on the same diagram is the layer interface as defined by the seismic survey. The cause of the apparent discrepancy is the variability of the weathered siltstone zone underlying the soil. Within this zone, the rock quality index (RQD)[1] was nearly 0 percent down to the approximate depth of the interface defined by the seismic survey, i.e., an average depth of 9 feet. This indicates a broken rock transition zone approximately 3 to 4 feet thick; the density of this material increased from 130 to 140 pcf with depth.

The subsurface borings indicated that the sandstones and other rocks below the weathered zone had high RQD values ranging from 90 to 100 percent. Figure 2.7 shows plots of RQD versus depth obtained from several borings as well as the average seismic velocity-depth profile. The RQD data indicate that below a depth of about 10 feet, the materials

[1] RQD is defined as 100 times the ratio of total length of core pieces greater than 4 inches long to total length of drive. It is used as an engineering index to the quality of the in situ rock.

are not badly fractured and that, if an open joint system consisting of nearly horizontal planes exists, the joints are not closely spaced. It should be noted that cores of the material could, however, easily be pulled apart along thin clay laminations. Thus, tensile strengths of the material in the vertical direction are very low.

One boring, U2, was drilled to a depth of 150 feet. The somewhat erratic variation of material type and composition properties extended to a depth of about 70 feet where a more uniform fine-grained sandstone of a lower dry density, i.e., 120 pcf, was encountered. This material is believed to be from the Wingate Formation. Because the samples were air-dried, no information on the in situ water content could be obtained for this material.

In June 1972, the plan for grading along the main WES gage line was received. It called for cut and fill in such a manner that the overburden thickness would be approximately 5 feet along the entire line. The density and water content of the fill were specified to match the in situ condition of the soil at a depth of 2-1/2 feet. Field density data obtained in June and July 1972 indicated that the average wet density and water content were 111 pcf and 7 percent, respectively. Figure 2.8 is a sketch of the idealized profile along the main WES gage line showing the filled section of varying thickness which was constructed prior to Event III and the underlying nonhomogeneous sedimentary materials.

At the time of Event III, the surface soil at the site was muddy due to wet weather conditions. Water contents were obtained from samples of the fill along the main gage line on the day before and the day after the event. The water contents are plotted versus distance from GZ in Figure 2.9. The results shown in Figure 2.9 indicate that the water content of the near-surface soil increased due to wet weather conditions from a value of about 7 percent as measured during the previous summer (and used to develop the preshot representative properties) to a value of about 15 percent at shot time. NX-size cores of the near-surface rock were taken at two locations to a depth of approximately 30 feet on the day following Event III. Water content and wet density

measurements determined from these cores appeared to be within the scatter of the original data, as shown in Figure 2.10.

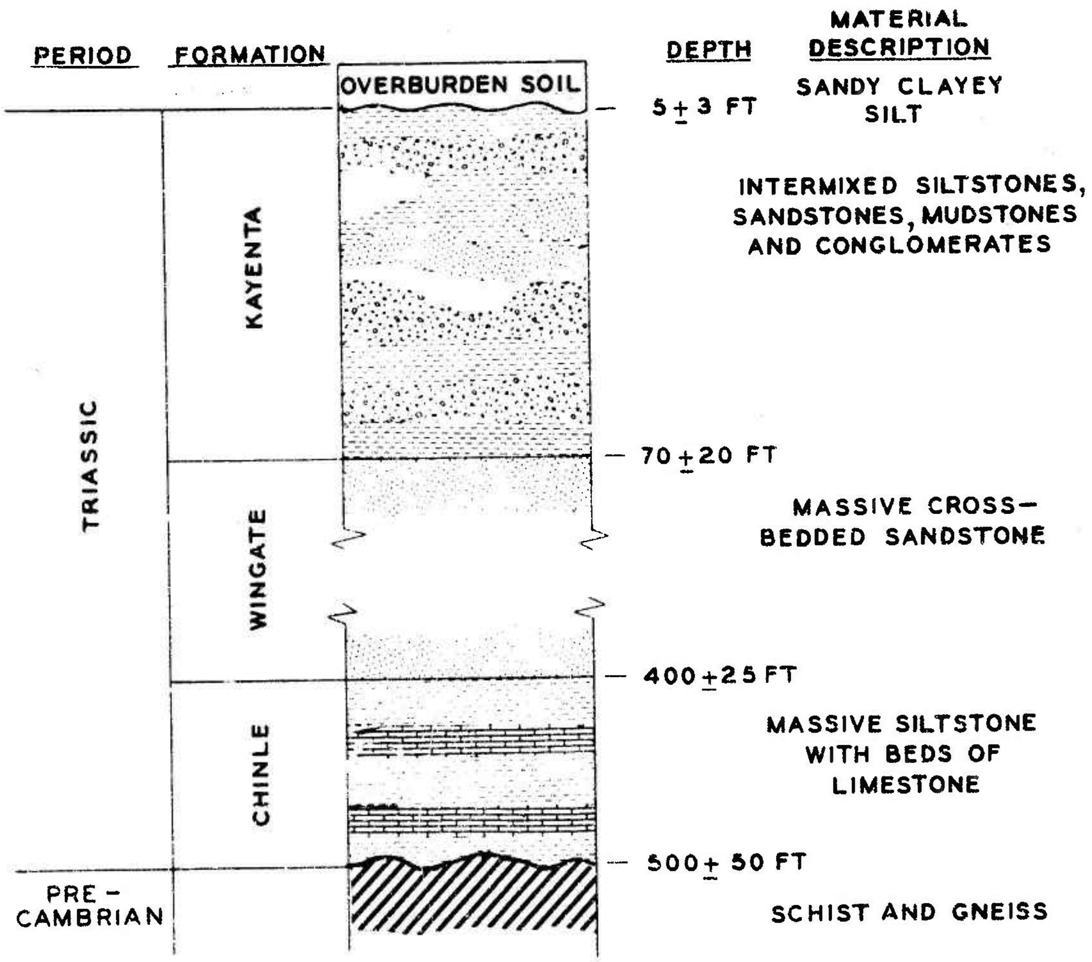

Figure 2.1 Stratigraphy column for Mixed Company site.

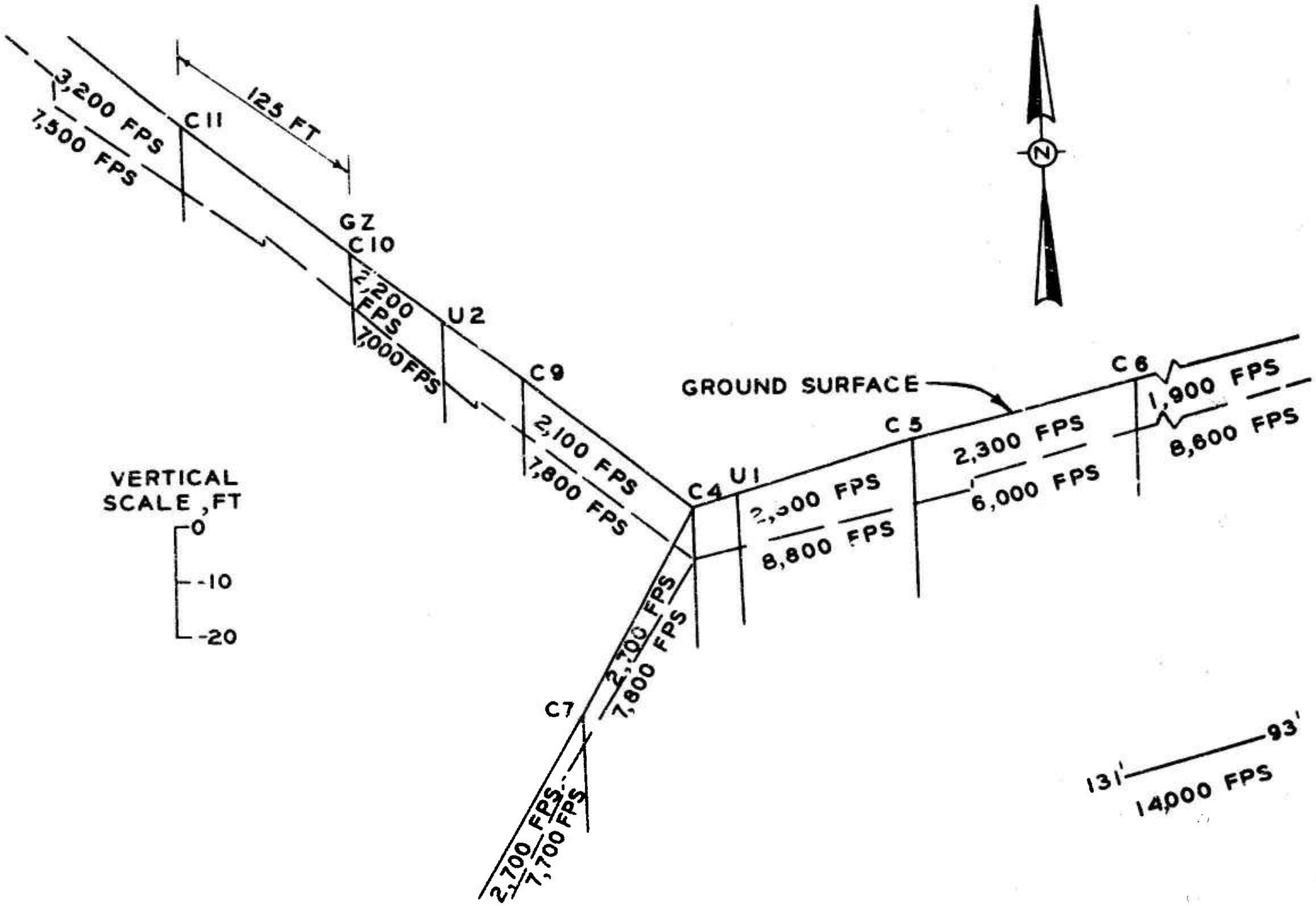

Figure 2.2 Schematic fence diagram along boring lines showing subsurface profile as defined by field refraction seismic survey.

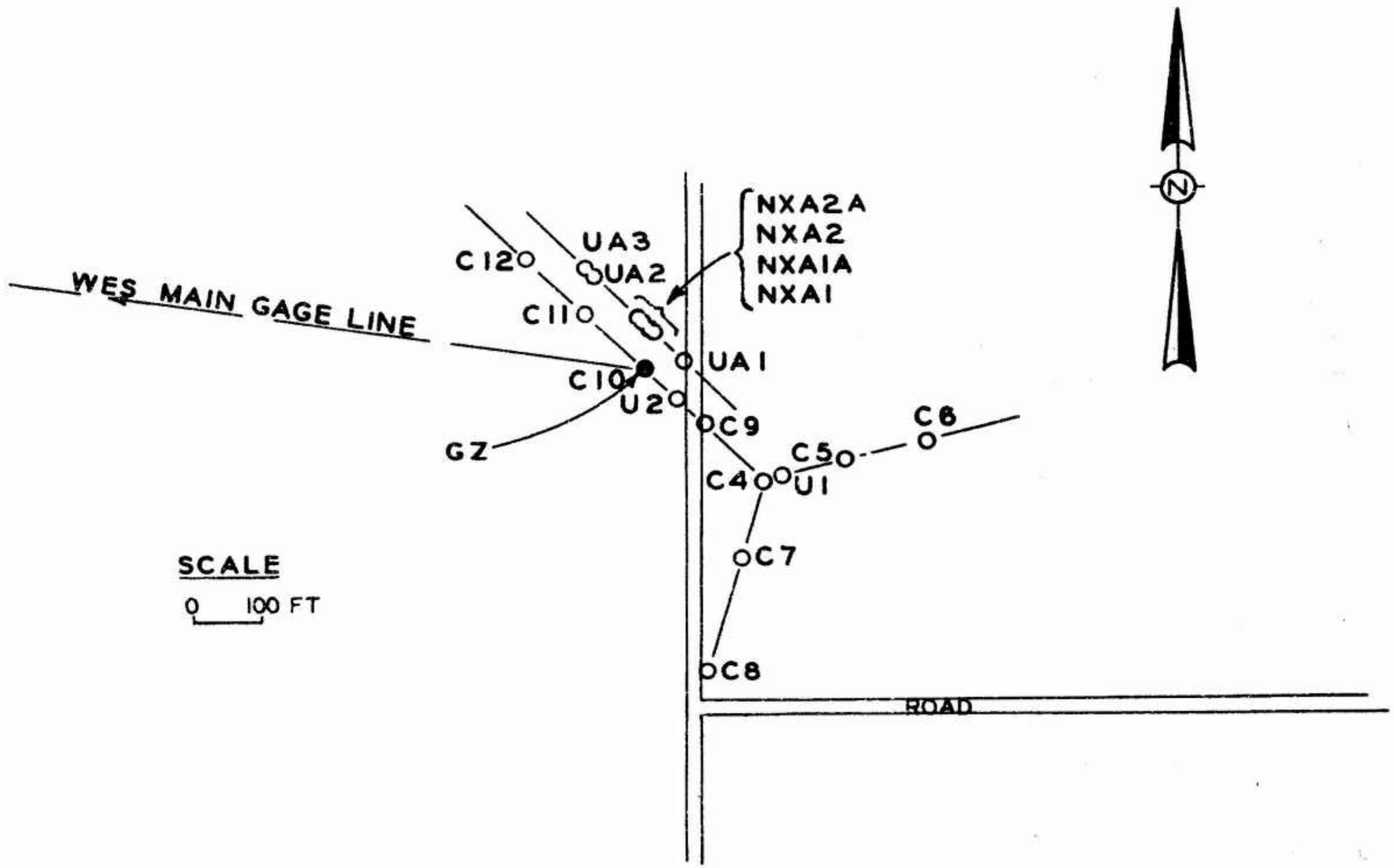

Figure 2.3 Plan view of the Mixed Company Event III site showing boring locations.

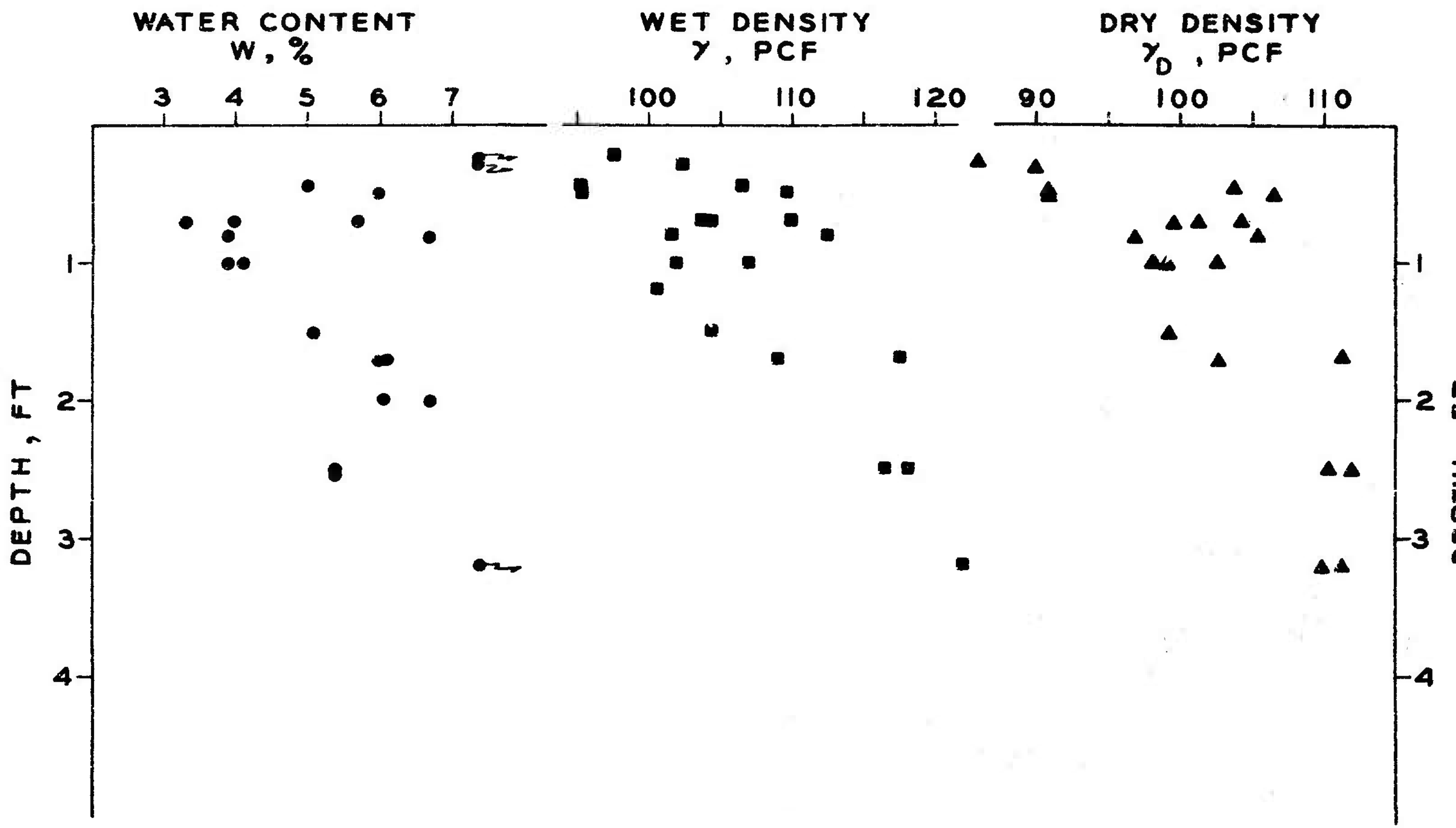

Figure 2.4 Composition properties of overburden soil zone.

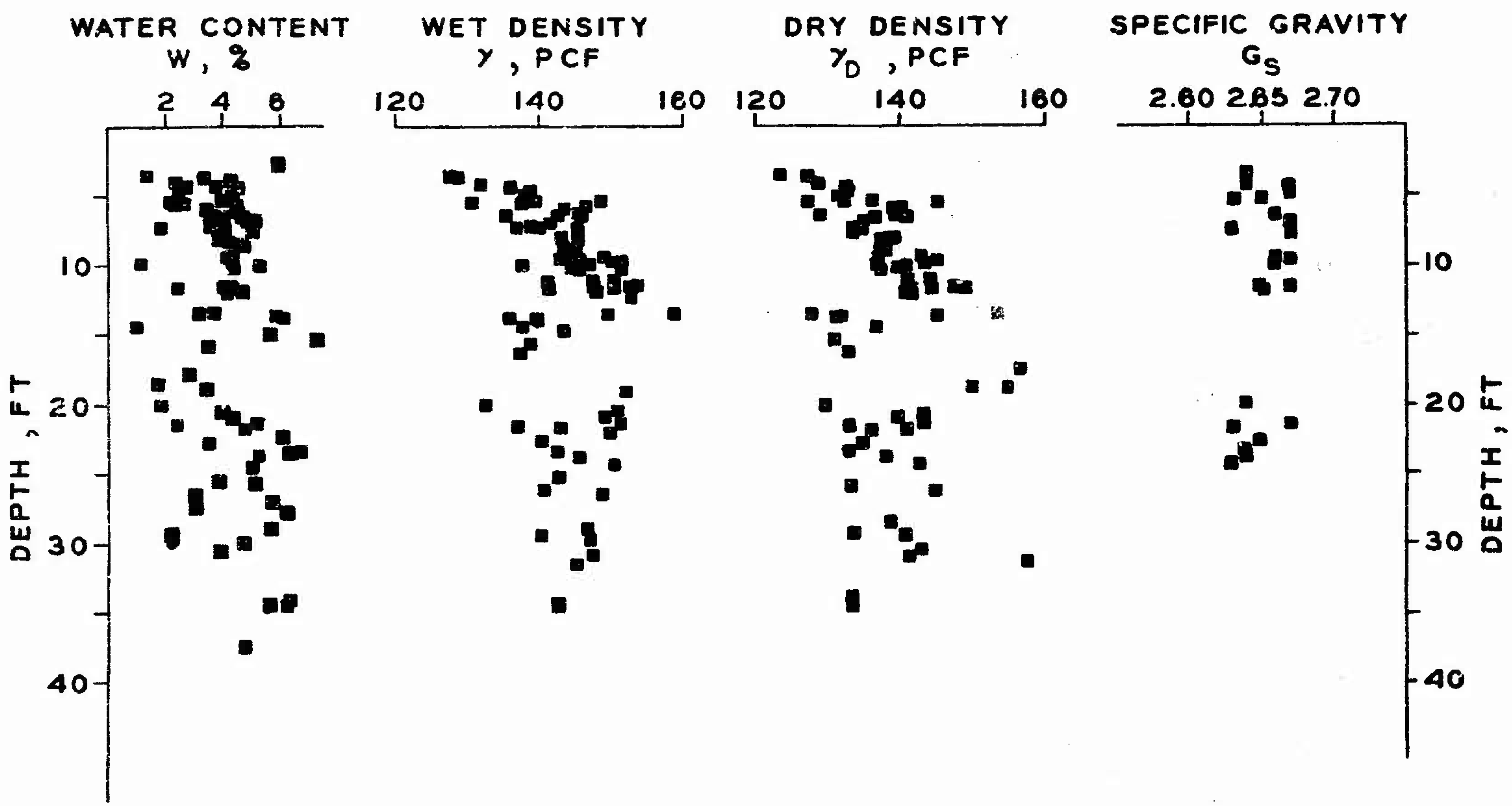

Figure 2.5 Composition properties of Kayenta Formation rocks.

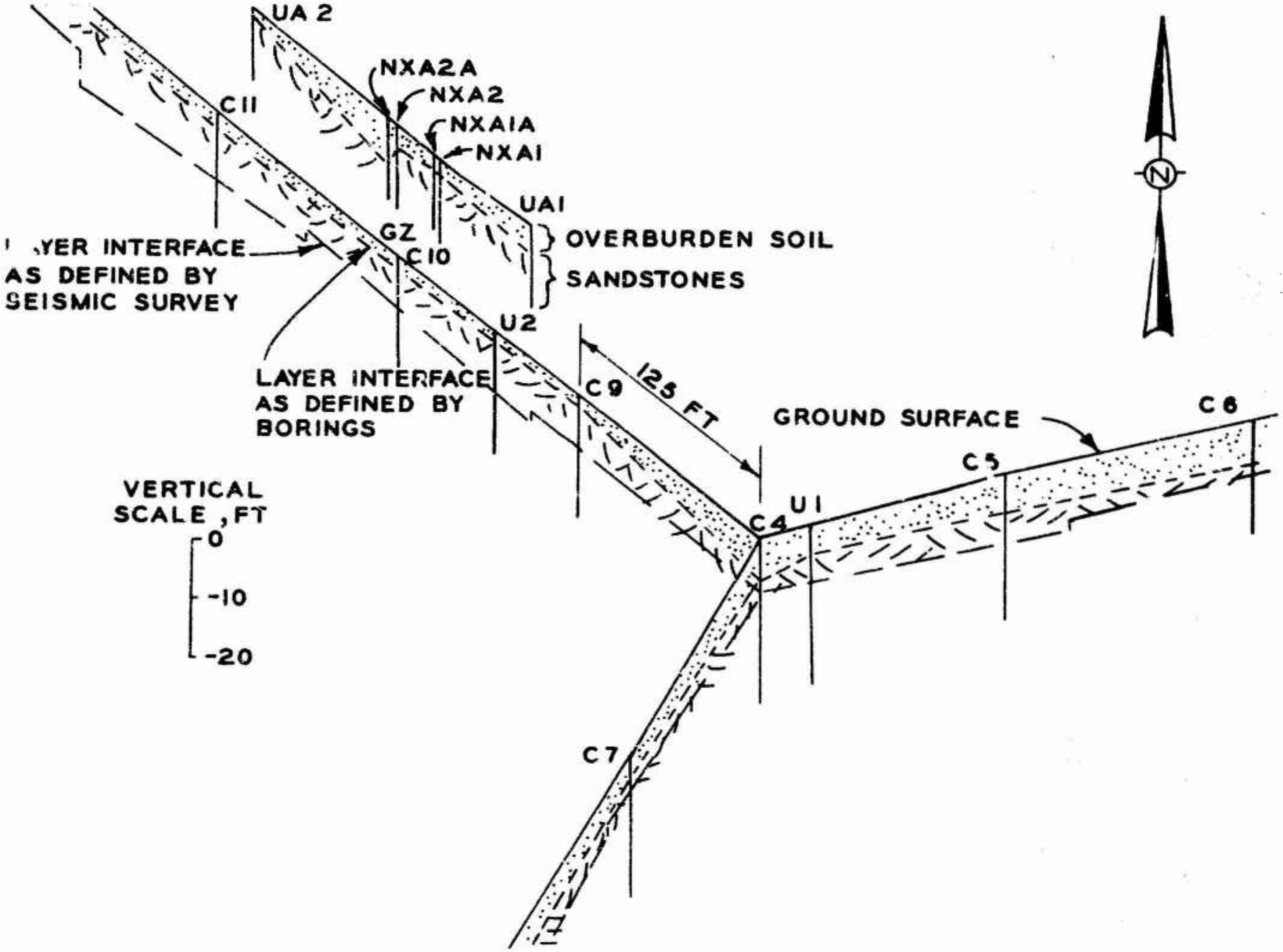

Figure 2.6 Schematic fence diagram showing the soil-rock interface as defined by borings compared with the interface defined by the seismic survey.

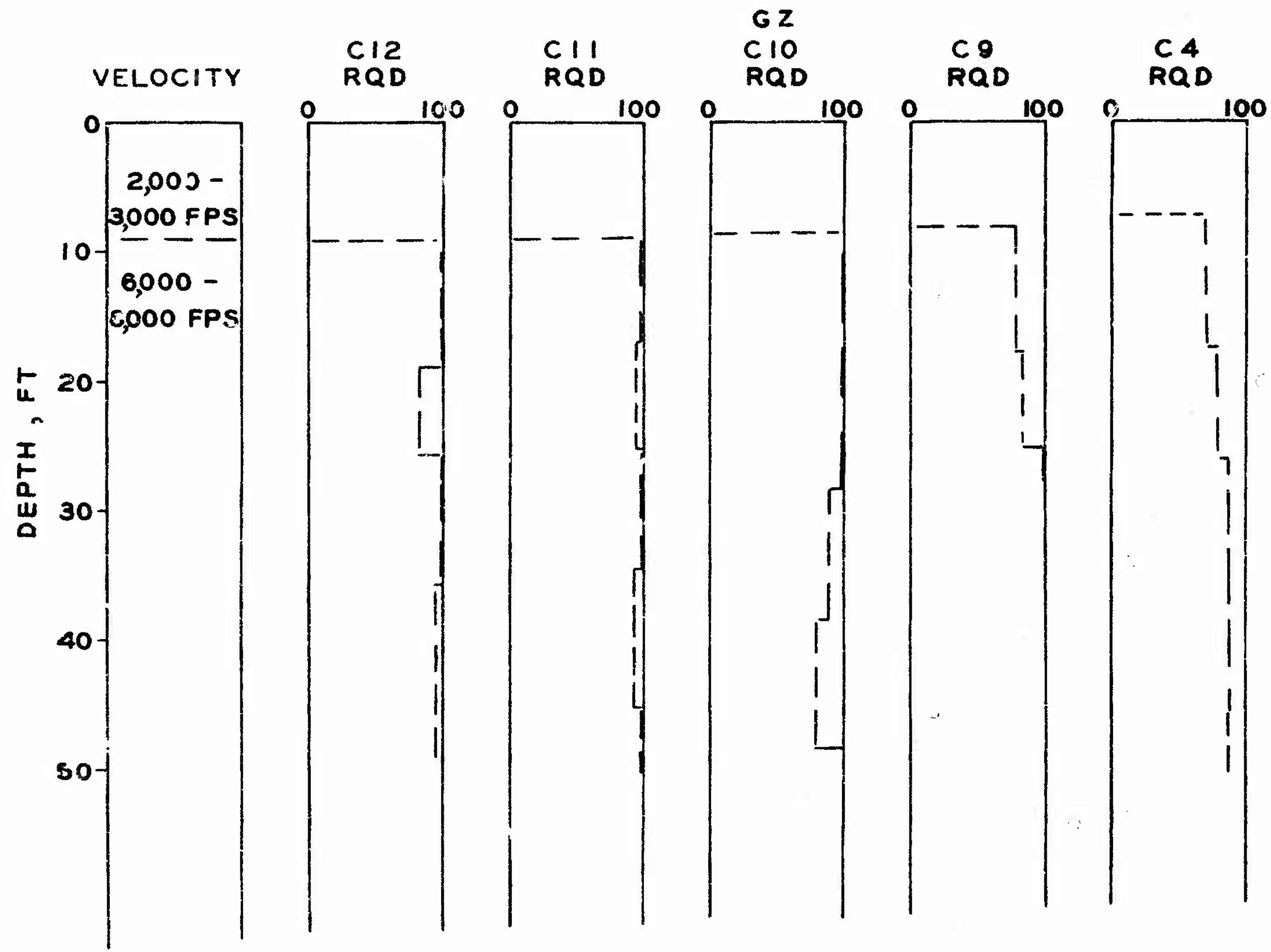

Figure 2.7 Rock quality index (RQD) of several borings compared with field velocity data.

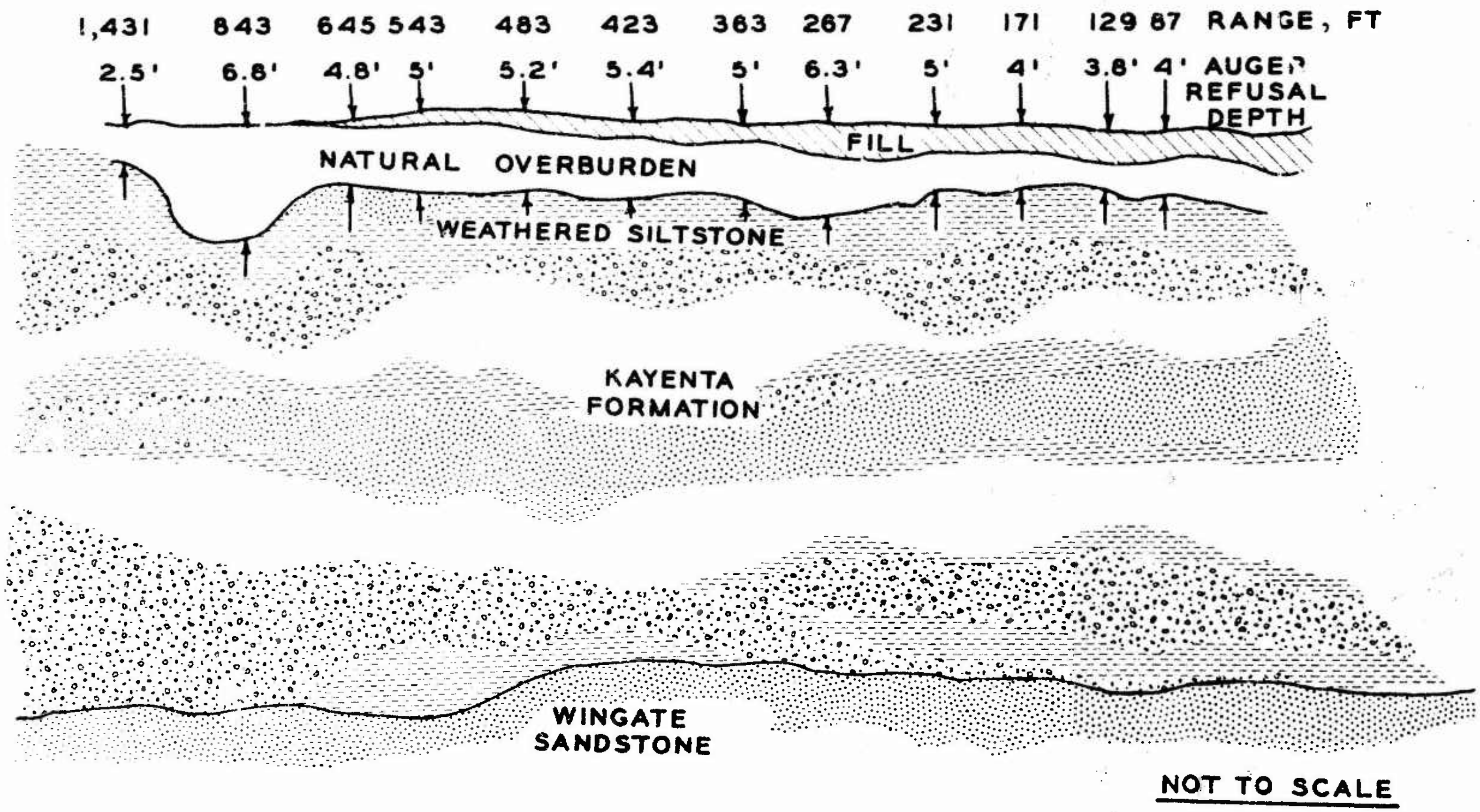

Figure 2.8 Idealized subsurface profile along main gage line indicating variation of materials.

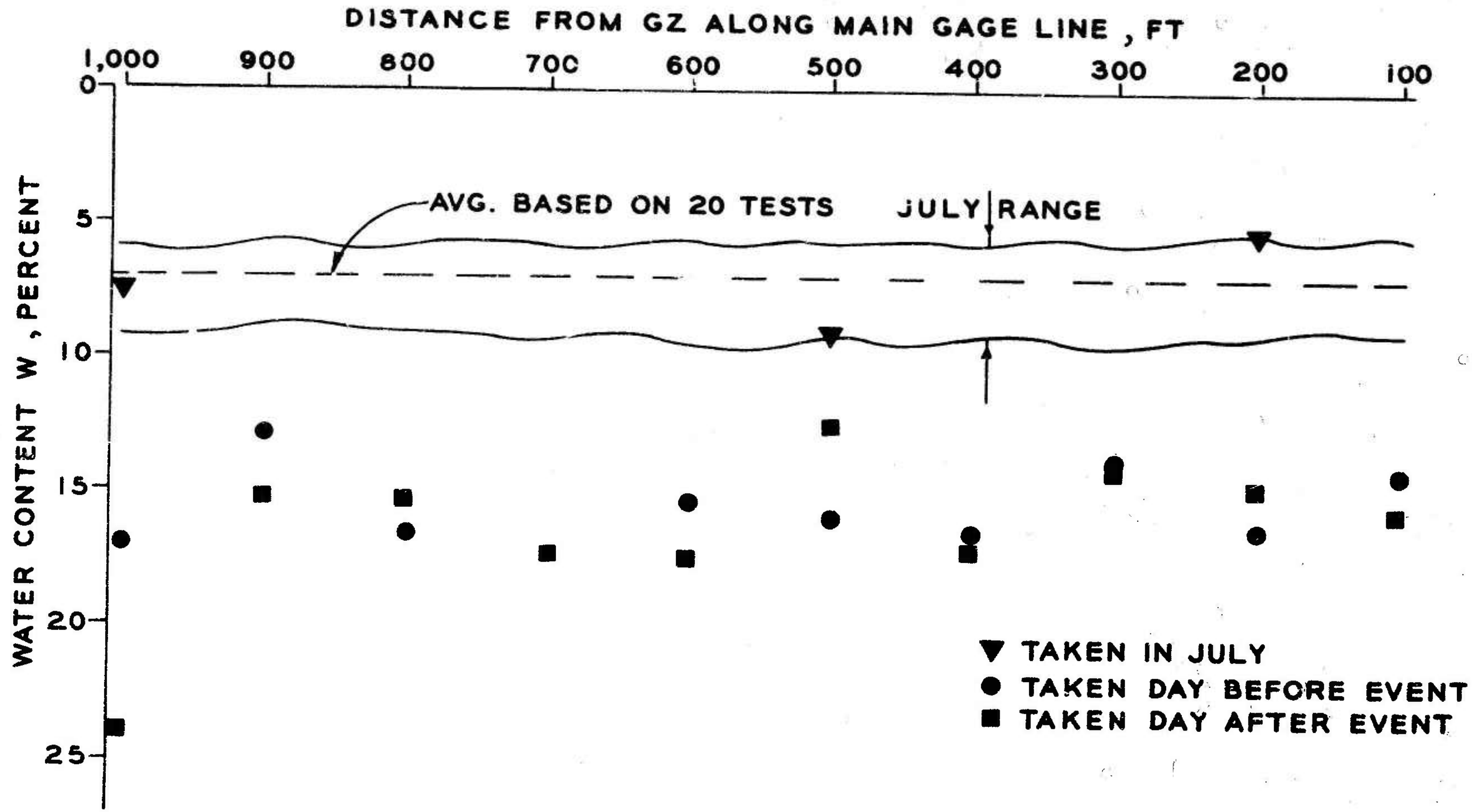

Figure 2.9 Water content of soil taken in July 1972 compared with water contents the day before and the day after Event III.

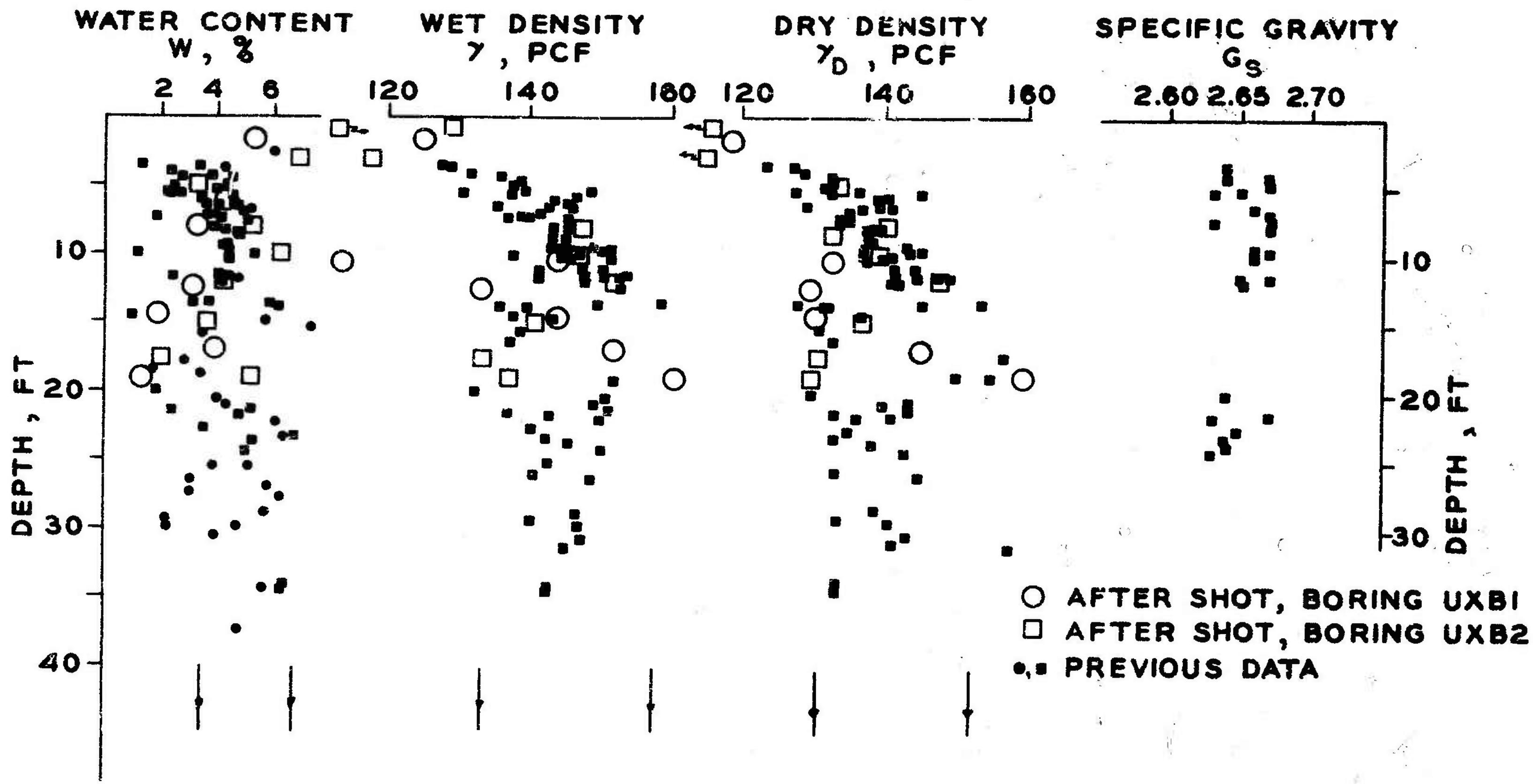

Figure 2.10 Original composition properties of Kayenta Formation and composition properties taken the day after Event III.

CHAPTER 3

LABORATORY TEST RESULTS

The objective of the laboratory test program was to investigate the responses of the Mixed Company III materials under several controlled states of stress. The properties were to be used in code calculations of the ground motion along the main WES gage line. Expected peak stress levels of interest in the field event varied from the kilobar range down to tenths of a bar; the response of the material was, therefore, examined over the regime.

Several laboratories participated in the test program. Each examined a given stress range and/or special aspect of the materials' responses. These laboratories were the WES Soils and Pavements Laboratory (S&PL), the WES Concrete Laboratory (CL), TT, and SRI. Each conducted tests on samples obtained from the borings made in September 1971 and/or February 1972. Two sizes of samples were available: NX-diameter (2-1/8 inch) and 5-inch diameter. Data from the tests conducted on the NX-size samples obtained in September 1971, however, were considered suspect since, as previously mentioned, those samples were air-dried in the field.

Since the purpose of this chapter is to present an overview of the results, data will be presented only to illustrate typical static and dynamic stress-strain response characteristics of the overburden and of the rock. The entire body of test data will be documented later.

3.1 TESTS

The tests conducted were grouped into four classes: (1) uniaxial strain (UX), (2) isotropic compression (IC), (3) triaxial shear (TX), and (4) tension tests. Three types of UX tests (i.e., radial strain, $\epsilon_r = 0$) were conducted: (1) gas or powder gun tests by CL and SRI in which a very thin wafer specimen was loaded by a flat-ended projectile with microsecond range loading rates to obtain stress-particle velocity relations, (2) tests by S&PL in which a 1- or 2.5-inch-high by 5-inch-diameter specimen was loaded by fluid pressure within loading rates

ranging from several milliseconds (dynamic) to several minutes (static) to obtain stress-strain relations, and (3) tests by S&PL, CL, and TT on cylindrical specimens statically loaded in the vertical direction through steel end caps while a circumferential or radial fluid pressure was applied to the specimen to restrain radial movement (detected by a sensor). This particular type of UX test is frequently referred to as a K_o test and yields both axial stress-axial strain and axial stress-radial stress relations.

The IC tests by S&PL, CL, and TT were conducted on cylindrical specimens loaded by a uniform all-around pressure to obtain pressure-volumetric strain relations. TX tests by S&PL, CL, and TT were conducted immediately following the loading phase of the IC tests. The hydrostatic pressure was held constant and the vertical axial load was increased until shear failure occurred. From the TX tests, stress difference-strain difference or stress difference-axial strain relations were obtained. The tensile tests by CL were conducted on cylindrical specimens either by direct pull along the vertical axis or by a line loading applied along the sides of the specimens. This latter type tension test is referred to as a Brazil test. In all of the tests, the specimens remained undrained during loading.

Table 3.1 presents a summary of the laboratory test program; it lists the performing laboratory, the type of test, the stress or pressure range investigated, the type material, and the approximate number of tests conducted. Included in the list are only those tests which were conducted prior to June 1972 and whose results were available for the development of the preshot constitutive property recommendations.

3.2 OVERBURDEN SOIL

The Mixed Company soil was classified as a sandy clayey silt (ML) in accordance with the Unified Soil Classification System. Since fill was to be added along the gage line and since the natural and fill overburden were to be confined into a single layer in the 2D calculations, response of this layer would be a function of the properties of both the natural soil and the compacted fill. Therefore, the soil was tested

in both the undisturbed condition and after it had been remolded to a range of densities and water contents.[1] The tests on the soil consisted of UX, IC, and TX tests. Figure 3.1 shows a plot of axial stress versus axial strain for several UX tests on the remolded soil at various densities. In general, the loading stress-strain curve stiffened with increasing density at a constant water content and the initial modulus softened with increasing water content at constant density.

The material indicated a strain-hardening response in shear with compaction occurring under low confining pressure (0 to 2,000 psi). The TX failure strength increased with increasing confining pressure. Some rate effects were noted, but they appeared to be masked by the effects due to the natural variation in density. Nevertheless, UX and TX tests with millisecond loading rates rather than static loading rates were used as the primary basis for development of representative properties since millisecond-range loading times were expected to occur in the actual field event.

3.3 WEATHERED SILTSTONE

Only a limited number of laboratory tests were conducted on specimens from the weathered siltstone zone due to the broken or laminated condition of the core. Most of the NX-size core pieces from this zone were less than 2 inches long and could be used only for water content and density measurements. UX tests were conducted on the 5-inch-diameter, Shelby-tube-encased samples. The steel tubes were first X-rayed in an attempt to locate horizontal separations. Attempts were then made to obtain intact 1-inch- or 2-1/2-inch-long specimens. It was found that even with the use of this procedure, many of the specimens contained open horizontal laminations or planes of weakness. One 2-1/2-inch-long specimen which appeared to be in good condition separated during handling. The exposed faces contained an intricate root

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[1] Since the field backfill density and water content had not been specified at the time of testing, a range of probable values had to be investigated.

system resembling a loosely woven piece of cloth. As a result, most of the UX specimens tested strained several percent in the vertical direction under very low stress levels (i.e., less than 10 psi). Above these levels, the UX curves stiffened gradually to approximately 1,000 psi, and thereafter a marked stiffening of the stress-strain curves occurred. Unloading moduli were quite stiff, resulting in significant hysteresis or inelastic response.

Several TX tests were conducted on specimens obtained from near the bottom of the weathered zone; results of these tests were similar to those of tests on specimens of the deeper sandstones, which will be presented later in this chapter. No direct-pull tension tests were conducted since the material would separate along the laminations under its own weight.

3.4 UNWEATHERED KAYENTA MATERIALS

The fact that a large variety of sandstones, siltstones, conglomerates, and mudstones appeared to occur randomly with depth within the Kayenta Formation made it desirable that a number or UX and TX tests be conducted on each material encountered if a reasonably accurate understanding of the overall behavior of the composite mass was to be achieved. Unfortunately, it was not possible to obtain representative test specimens in all cases; therefore, parametric studies had to be conducted. For example, since some of the available NX-size cores had been air-dried, it was necessary to determine the effects of water content variations on the UX and TX behavior. In addition, static and dynamic UX, IC, and TX tests were conducted to determine sensitivity to loading rate effects.

<u>3.4.1 UX Gun Tests</u>. Gas or powder gun tests were conducted on both wet and dry specimens from the Kayenta Formation. Although most of the testing was limited to the better quality sandstones, some tests were performed on other materials. Attempts were also made to replace water loss during specimen preparation. Typical test results, as shown in Figure 3.2, indicated a stiff initial stress-strain relation, followed by a softening associated with structural collapse of the cemented

grains. The level of the softening or collapse varied from 4 to 6 kbar. This limited data did not indicate a difference in the stress level associated with collapse between the dry specimens and the partially saturated specimens. However, scatter in the gun test data was noted, but it appeared to be due to differences between the various test specimens in lithology and density. Two tests were conducted on similar air-dried specimens at stress levels above 20 kbar, and these results are also shown in Figure 3.2. The data indicated a stiffening at strain levels approximately equal to the volume percentage of air V_a within the specimen. The inserts shown in Figure 3.2 indicate different possible interpretations of the data with regard to the unloading path from those stress levels.

3.4.2 UX Tests on 5-Inch-Diameter Specimens. Both static (several minutes to peak stress) and dynamic (several milliseconds to peak stress) tests were conducted. No differences greater than the general data variation or scattering for a given material were noted between the data from dynamic tests and the data from static tests. Although most of these tests were conducted on specimens with preserved in situ water contents, tests were also conducted on laboratory air-dried specimens and on specimens which were rewetted after first being air-dried. The results did not indicate effects greater than the general data variation.

UX tests to 200 psi were conducted to examine the initial loading moduli and the unloading response at low stress levels. Results, such as those shown in Figure 3.3, did not, in general, correlate well with the initial moduli associated with the field seismic velocity. However, they did indicate a significant hysteretic behavior of the materials even when unloading occurred from these low pressures.

Figure 3.4 shows typical UX stress-strain results from some of the 2,000-psi-level tests on the Kayenta materials from shallow (<25 feet) depths; typical results from the overlying weathered siltstone are shown for comparison. Because all of the lithologies encountered in the upper Kayenta Formation are represented in the Figure 3.4 test

results, which include results of tests on weathered specimens, the data scatter is quite large.

Stress-strain curves from four selected 8,000-psi-level UX tests, which bound all the 8,000-psi data shown in Figure 3.4, are shown in Figure 3.5. As in the case of the 2,000-psi-level results, the scatter in these higher stress level tests is also primarily a function of change in lithology and weathering. Again, it was not possible to establish a definite effect due to the loading rate effect between the dynamic (ten milliseconds to peak) and static tests on similar materials.

3.4.3 UX K_o Tests. Static K_o tests were conducted to determine the stress path response (principal stress difference versus mean normal stress) of the material. Peak stress levels ranged from about 0.1 to 4 kbar. The data indicated a low initial Poisson's ratio which increased slightly with stress. A marked decrease in the stress path slope (signifying a marked increase in Poisson's ratio) was noted at the 2- to 3-kbar principal stress difference level. This stress difference level is in general agreement with that at which structural collapse was observed in the UX gun test data. At higher stress differences, the stress path curvature reversed, i.e., the slope increased again, thereby indicating a decrease in Poisson's ratio. Figure 3.6 shows the K_o stress path data generated by TT and a typical representative UX stress path relation derived from them.

3.4.4 IC and TX Tests. Most of the IC tests were conducted as the confining pressure application phase of TX tests and were, therefore, really one part of a two-part test. Both static and dynamic IC tests were conducted to 8,000 psi and static tests were conducted at pressures up to 4 kbar. The general shapes of the pressure-volumetric strain curves were similar to those of the axial stress-axial strain curves from the UX tests. Figure 3.7 shows results of static and dynamic tests on sandstones from shallow (<25 feet) depths; it indicates approximately the same range of scatter in both static and dynamic loadings as was noted in the UX tests on similar materials. The IC tests to 4 kbar yielded pressure-volumetric strain curves which

continued to stiffen with increasing pressure, i.e., no structural collapse or softening of the pressure-volumetric stress relation was observed up to 4 kbar. The fact that collapse occurred during the UX loading at these stress levels and did not occur under hydrostatic loading indicates a dependence of the collapse phenomena on principal stress difference or shear stress.

A series of static and dynamic TX tests were conducted at confining pressures up to 6,000 psi. The principal stress difference versus axial strain relation indicated a dependence on confining pressure prior to failure; the failure strength of all materials increased with increasing confining pressure. Axial strains at failure for most of the Kayenta materials were on the order of 1 percent when tested under confining pressures less than 6,000 psi. However, a much softer response (i.e., greater strains) was noted on a clayey conglomerate found near the 20-foot depth. Figure 3.8 is a plot of principal stress difference versus axial strain showing representative dynamic shear test data for most of the materials tested including the clayey conglomerate.

A series of static TX tests was also conducted at confining pressures of 2 to 4 kbar. The initial loading moduli from these high confining pressure tests were similar to those in the lower pressure tests, and the failure strengths continued to increase with increasing confining pressures. But the strains at which failure occurred were significantly greater for the high-pressure tests, i.e., failure occurred at 5 to 10 percent axial strain, whereas in the lower confining pressure tests, failures generally occurred at about 1 percent axial strain.

Volumetric strain changes of the sandstones during shear were also observed during the testing. Although some compaction was noted during initial loading, at stress states near failure, the material dilated as shown in Figure 3.9. Upon loading, volumetric strain tended to reverse and recompaction occurred. After removal of shear loading, the unloading portions of the pressure-volumetric strain curves that were generated as the mean normal stress was subsequently removed were similar to the unloading curves from IC tests without follow-on TX shear phases. The dashed lines in Figure 3.9 are for clarity only.

The TX failure envelopes obtained for the sandstones showed surprisingly little variation considering the wide variation noted in UX and IC test data. Figure 3.10 is a plot of both static and dynamic failure data on sandstones from depths less than 25 feet. Several tests conducted at other than constant confining pressure conditions indicated that the strength of the material was independent of the loading path loading to failure.

3.4.5 Tension Tests. Two types of tension tests were conducted on the sandstones, i.e., direct-pull and Brazil tests. The direct-pull tests were conducted on a limited number of NX-size specimens which were oriented with their axis of symmetry in the vertical direction. Most of the failure stresses were less than 100 psi, and many specimens fell apart under their own weight along thin horizontal clay seams of laminations. The Brazil tests were conducted by applying a line load along the sides of a number of NX-size core pieces. There is some question as to the actual validity of the tests results, but it was felt that they could provide an index to the tensile strengths of the materials. Failure stresses as high as 500 psi were measured. Although the exact stress levels may be questionable, the results do indicate that the materials encountered have a substantially higher tensile strength in the horizontal direction than in the vertical direction.

TABLE 3.1 SUMMARY OF LABORATORY TEST PROGRAM

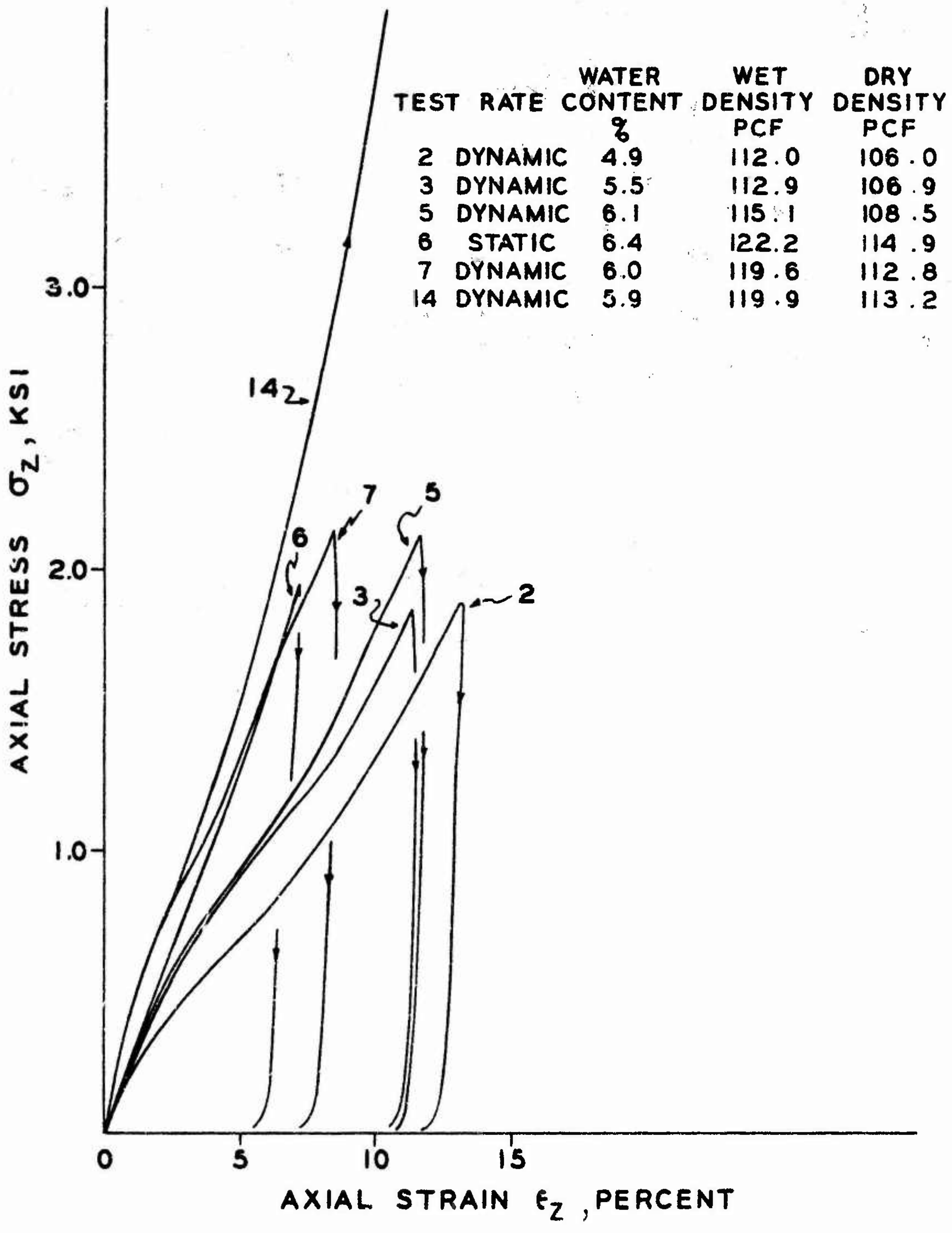
Laboratory	Type of Test[a]	Pressure Range	Type Material	Approximate No. of Tests
WES S&PL	Static and dynamic UX	Axial stresses to 10,000 psi	Undisturbed and remolded soil	22
	Static and dynamic TX	Confining pressures to 6,000 psi		37
	Static K_o	Mean pressures to 2,000 psi		7
	Static and dynamic UX	Loaded to 200 psi, unloaded, and reloaded to 2,000 psi	Rock	30[b]
	Static and dynamic UX	Axial stresses to 2,000 psi	Rock	10
	Static and dynamic UX	Axial stresses to 10,000 psi	Rock	19
	Static and dynamic IC	Mean pressures to 8,000 psi	Rock	50
	Static and dynamic TX	Confining pressures to 6,000 psi	Rock	43[c]
	Static K_o	Mean pressures to 2,000 psi	Rock	16
	Special stress path tests	Mean pressures to 10,000 psi	Rock	10
WES CL	Static IC	Mean pressures to 2,000 psi	Rock	5
	Static TX	Confining pressures to 2,000 psi	Rock	22[d]
	Static K_o	Mean pressures to 30,000 psi	Rock	2
	Static tension	Loaded to failure	Rock	23
	UX powder gun	Loaded to 70 and to 90 kbar	Remolded soil	2
	UX gas gun	Loadings from 5 to 10 kbar	Rock	10
	UX powder gun	Loaded to 30 and to 55 kbar	Rock	2
TT	Static IC	Mean pressures to 4 kbar	Rock	7
	Static TX	Confining pressures of 2 and 4 kbar	Rock	14
	Static TX	Confining pressures to 1/2 kbar	Rock	7
	Static K_o	Mean pressures to 4 kbar	Rock	3
	Special stress path tests	Mean pressures to 4 kbar	Rock	5
SPI	UX gas gun	Loadings from 5 to 10 kbar	Rock	5

[a] UX = uniaxial strain; TX = triaxial shear; K_o = UX test on cylindrical specimen; IC = isotropic compression test.

[b] Measurement system changed between loadings.

[c] Ten of the 43 tests were not conducted to failure.

[d] Includes three tests with confining pressures of 10,000 psi.



TEST	RATE	WATER CONTENT %	WET DENSITY PCF	DRY DENSITY PCF
2	DYNAMIC	4.9	112.0	106.0
3	DYNAMIC	5.5	112.9	106.9
5	DYNAMIC	6.1	115.1	108.5
6	STATIC	6.4	122.2	114.9
7	DYNAMIC	6.0	119.6	112.8
14	DYNAMIC	5.9	119.9	113.2

Figure 3.1 Results of UX tests on soil specimens remolded to different densities.

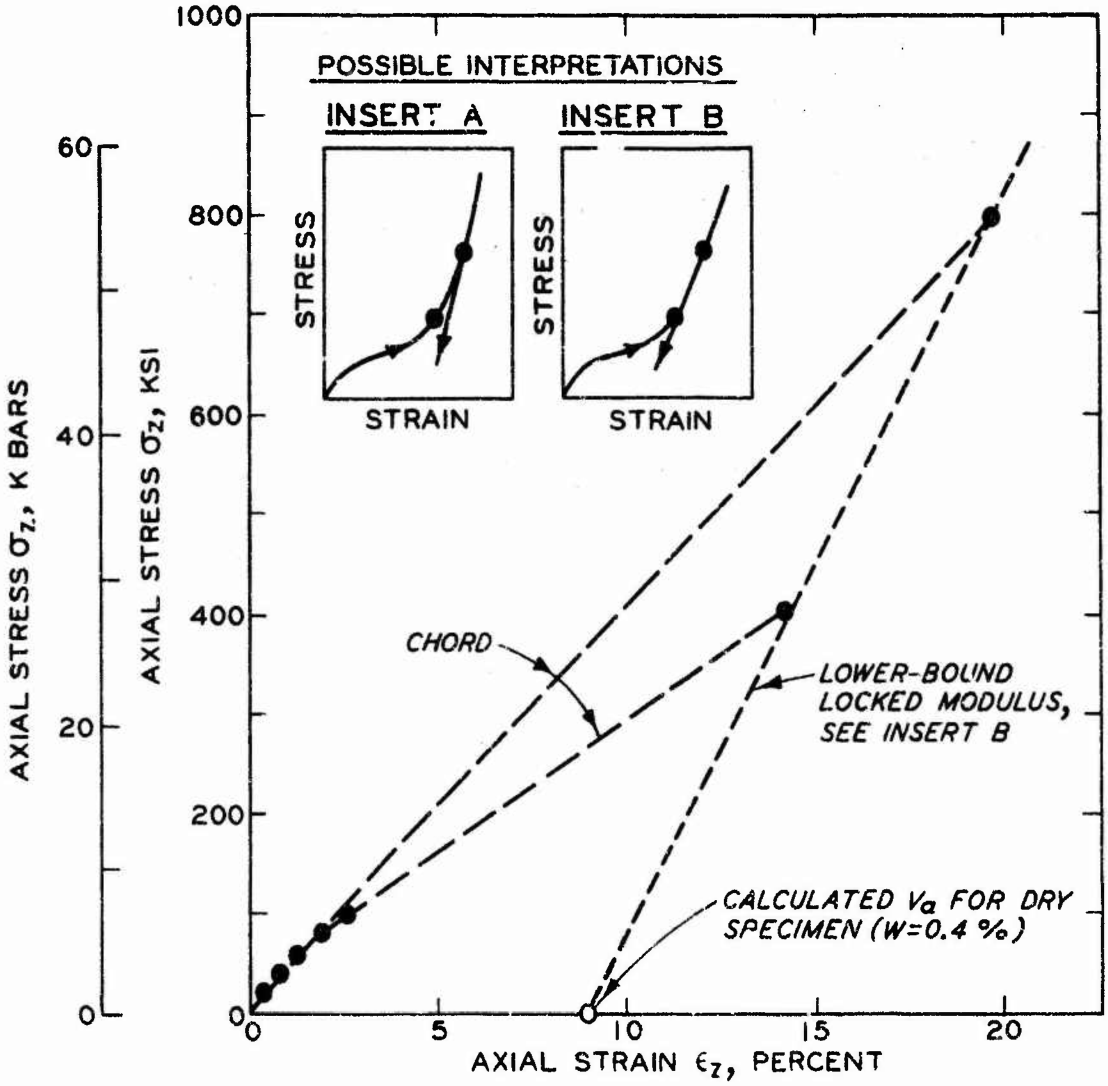


Figure 3.2 Results of UX gun tests showing response of air-dried sandstone to high pressures.

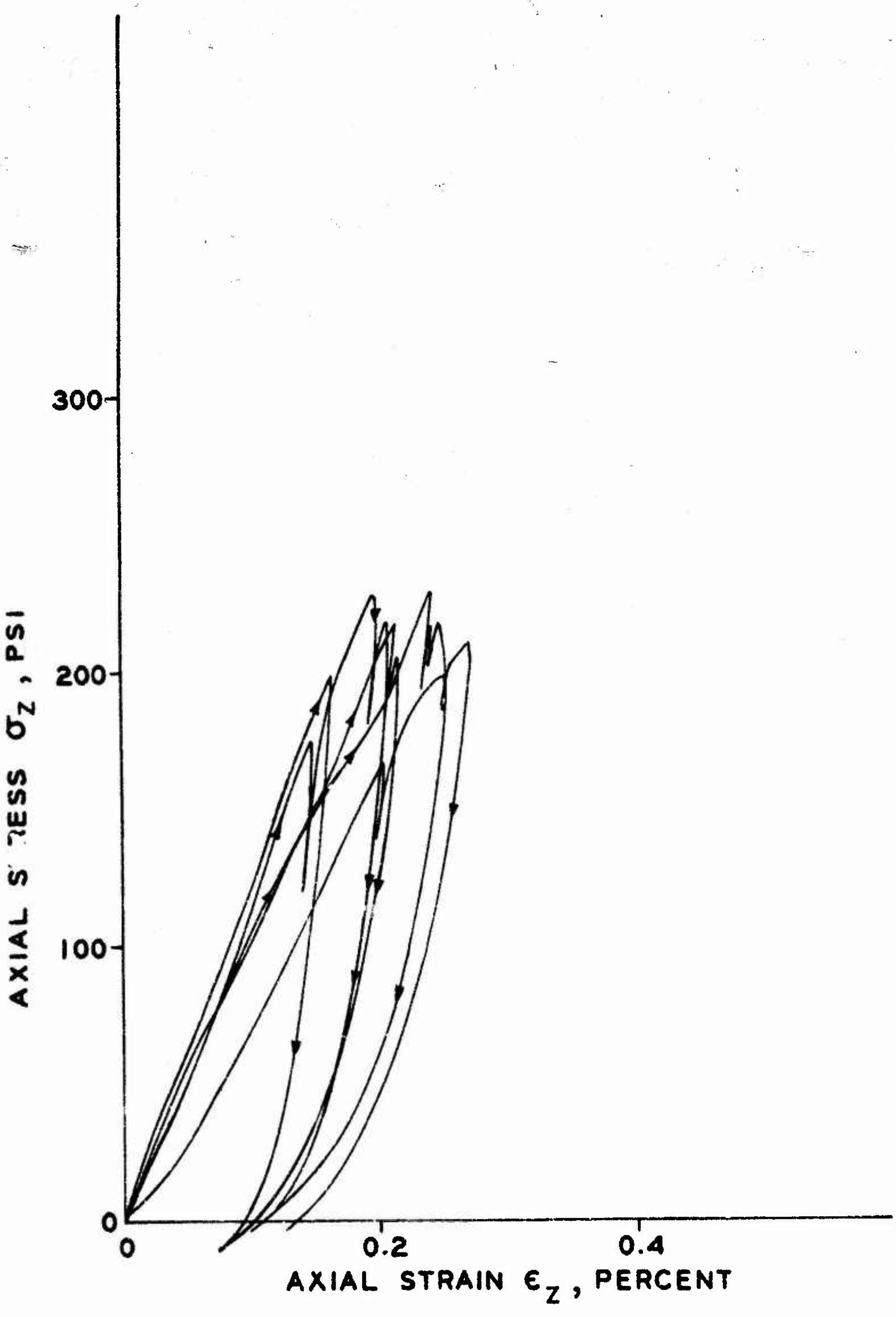


Figure 3.3 Typical dynamic UX test results for shallow (<25 feet) sandstones loaded to 200 psi.

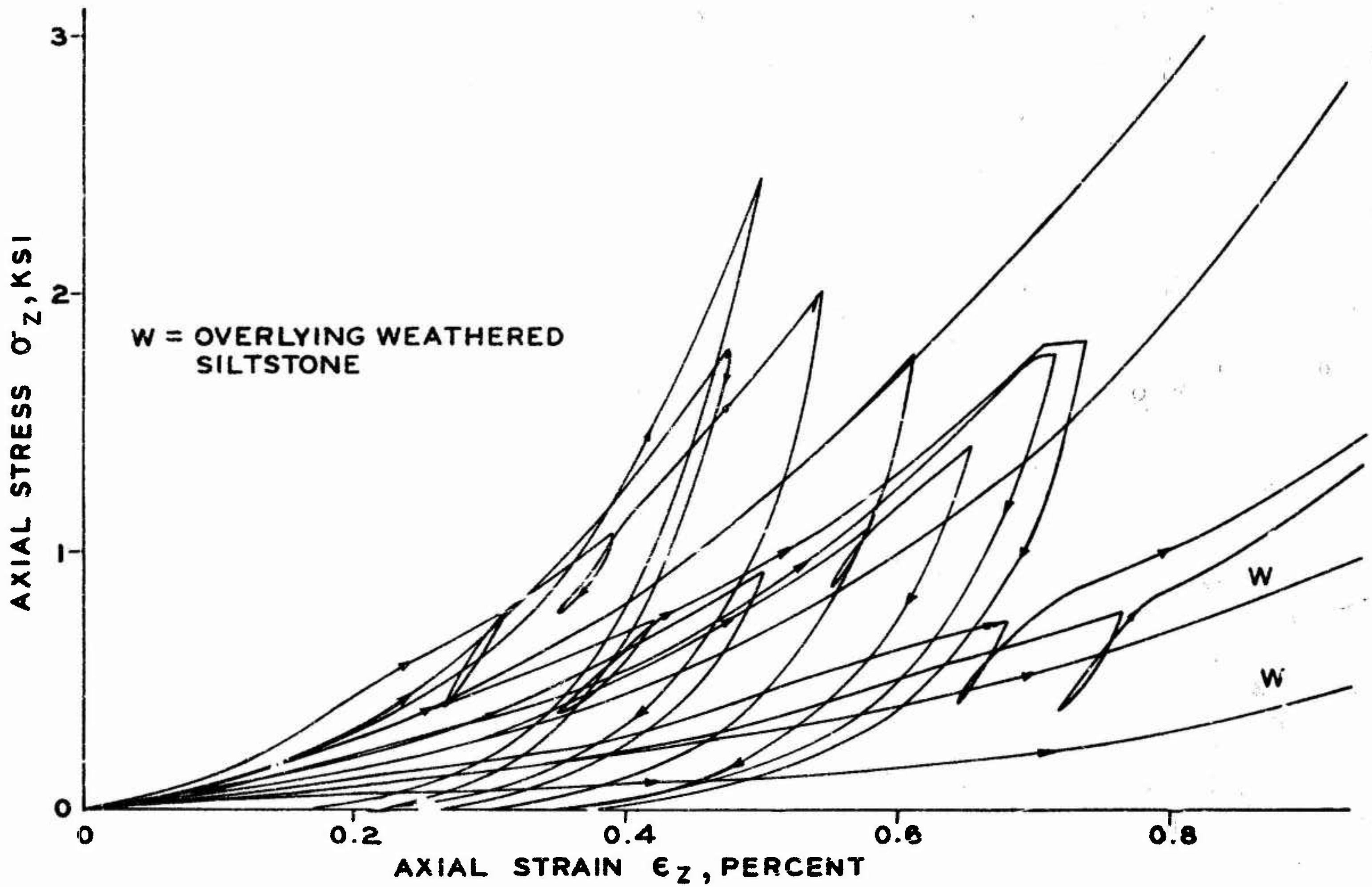


Figure 3.4 Typical dynamic UX test results on Kayenta materials from shallow (<25 feet) depths loaded to 2,000 psi.

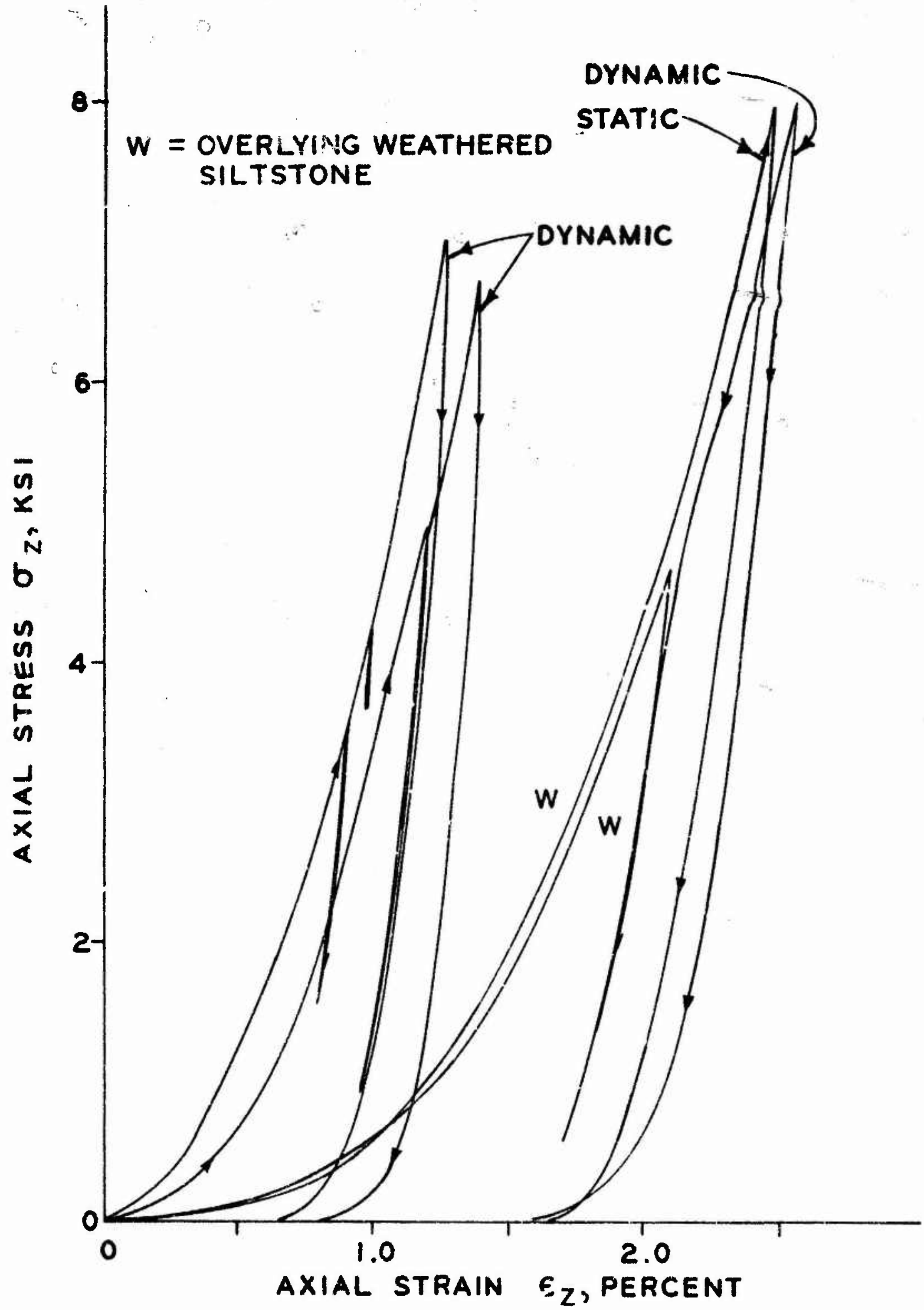


Figure 3.5 Selected dynamic and static UX test results on Kayenta materials from shallow (< 25 feet) depths loaded to 8,000 psi.

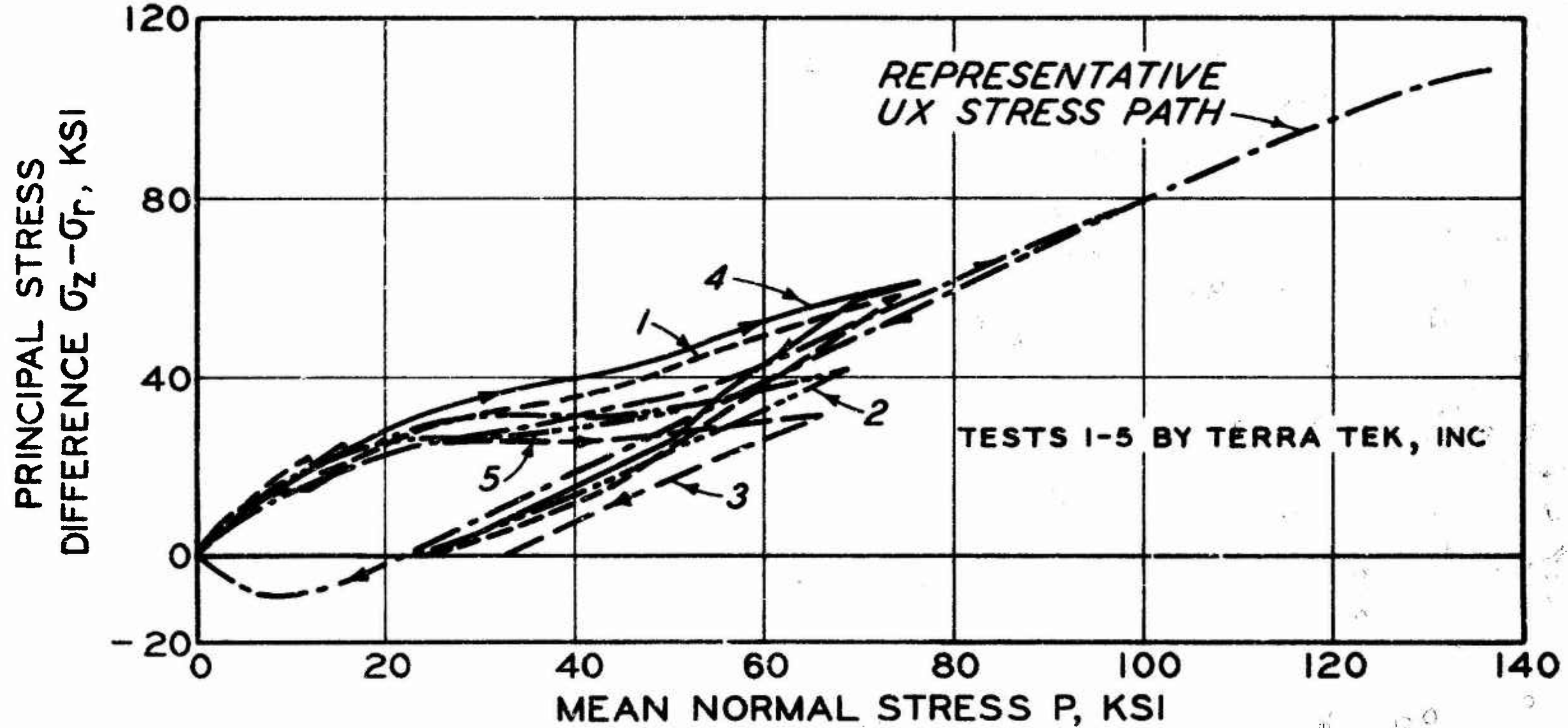


Figure 3.6 Static K_o test results on sandstones compared with representative UX stress path response.

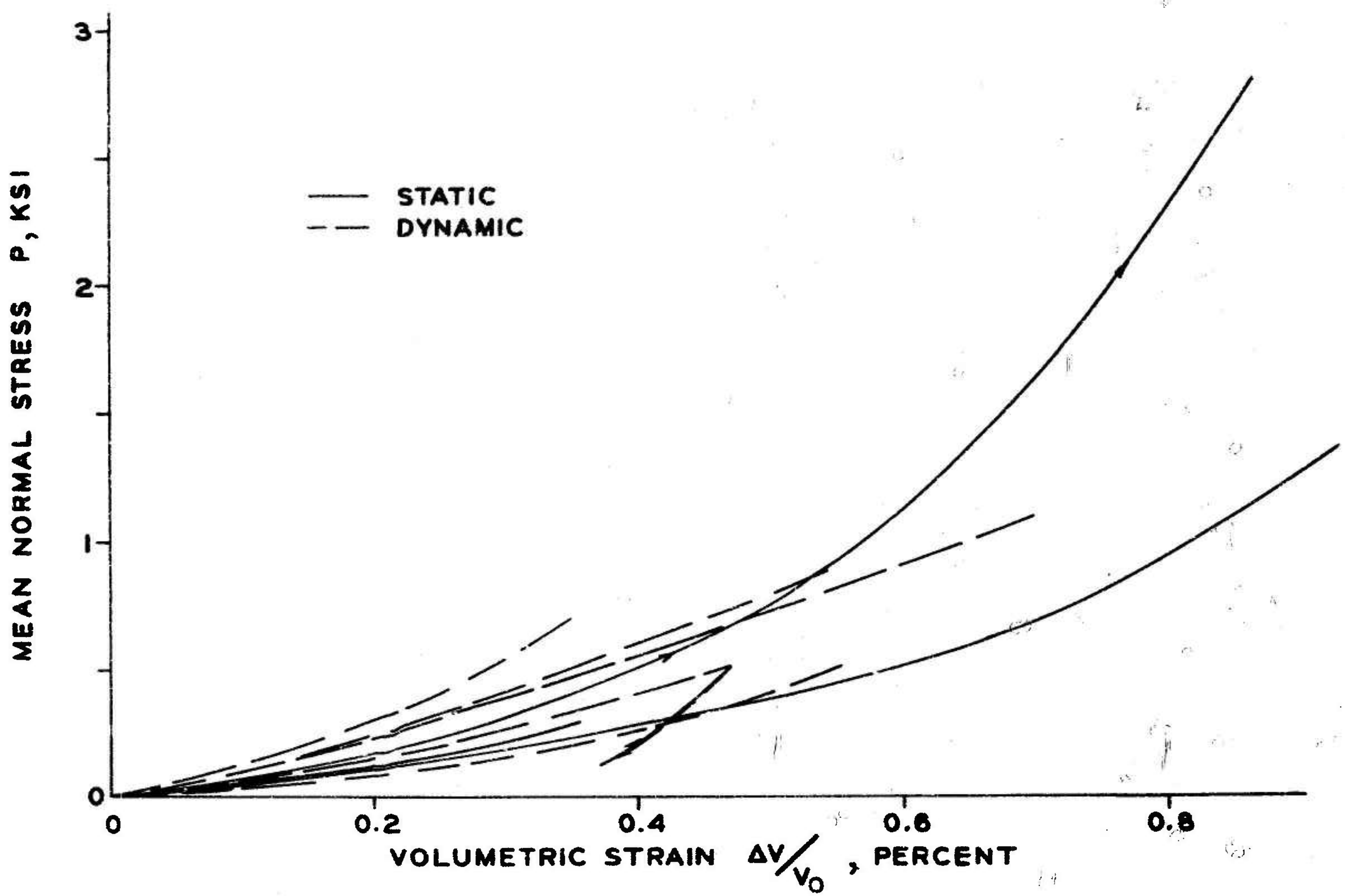


Figure 3.7 Typical static and dynamic IC test results on shallow sandstone (<25 feet).

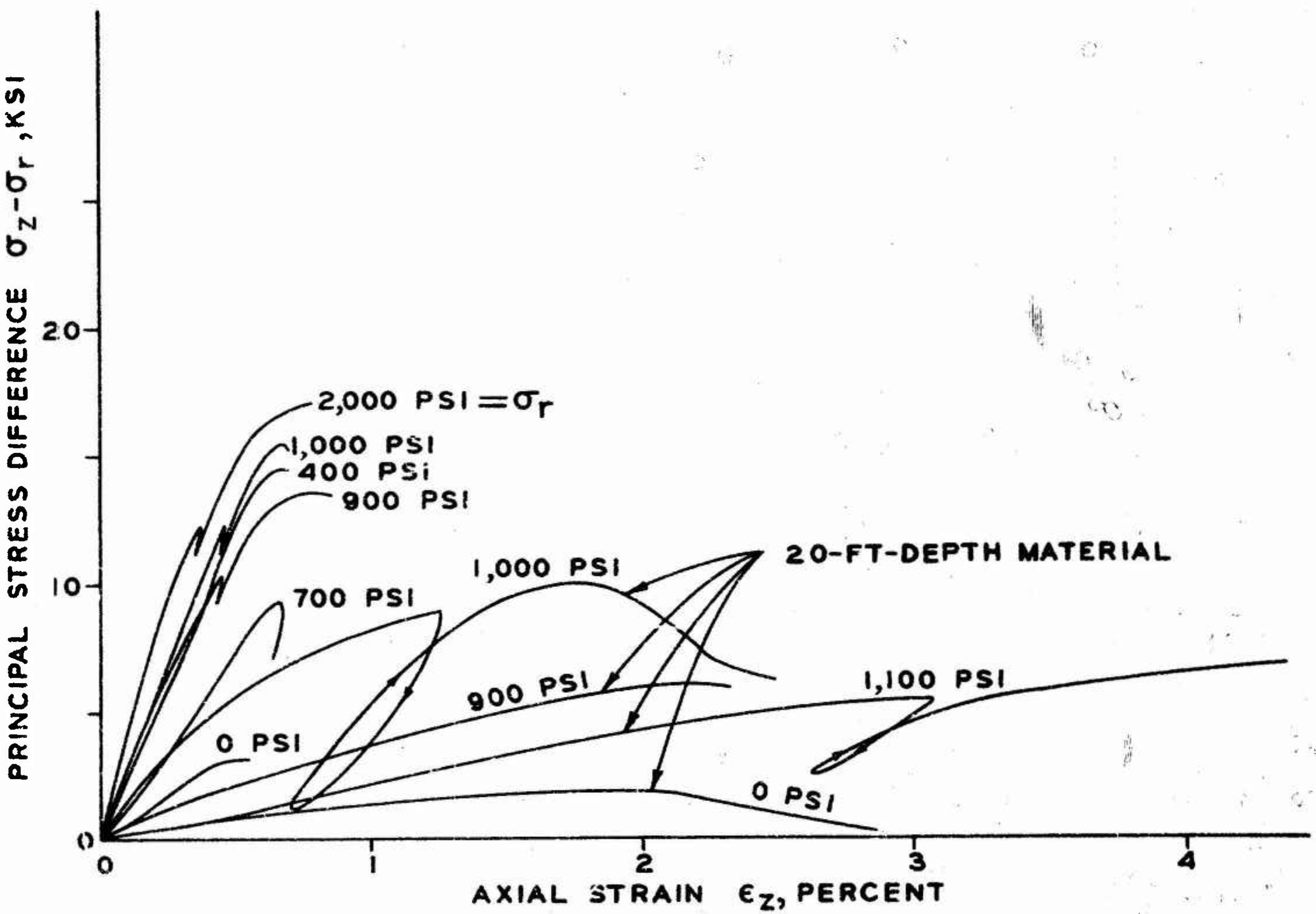


Figure 3.8 Typical dynamic TX shear test results at various confining pressures showing the response of the clayey conglomerate material (from 20-foot depth) and the general response of other Kayenta materials.

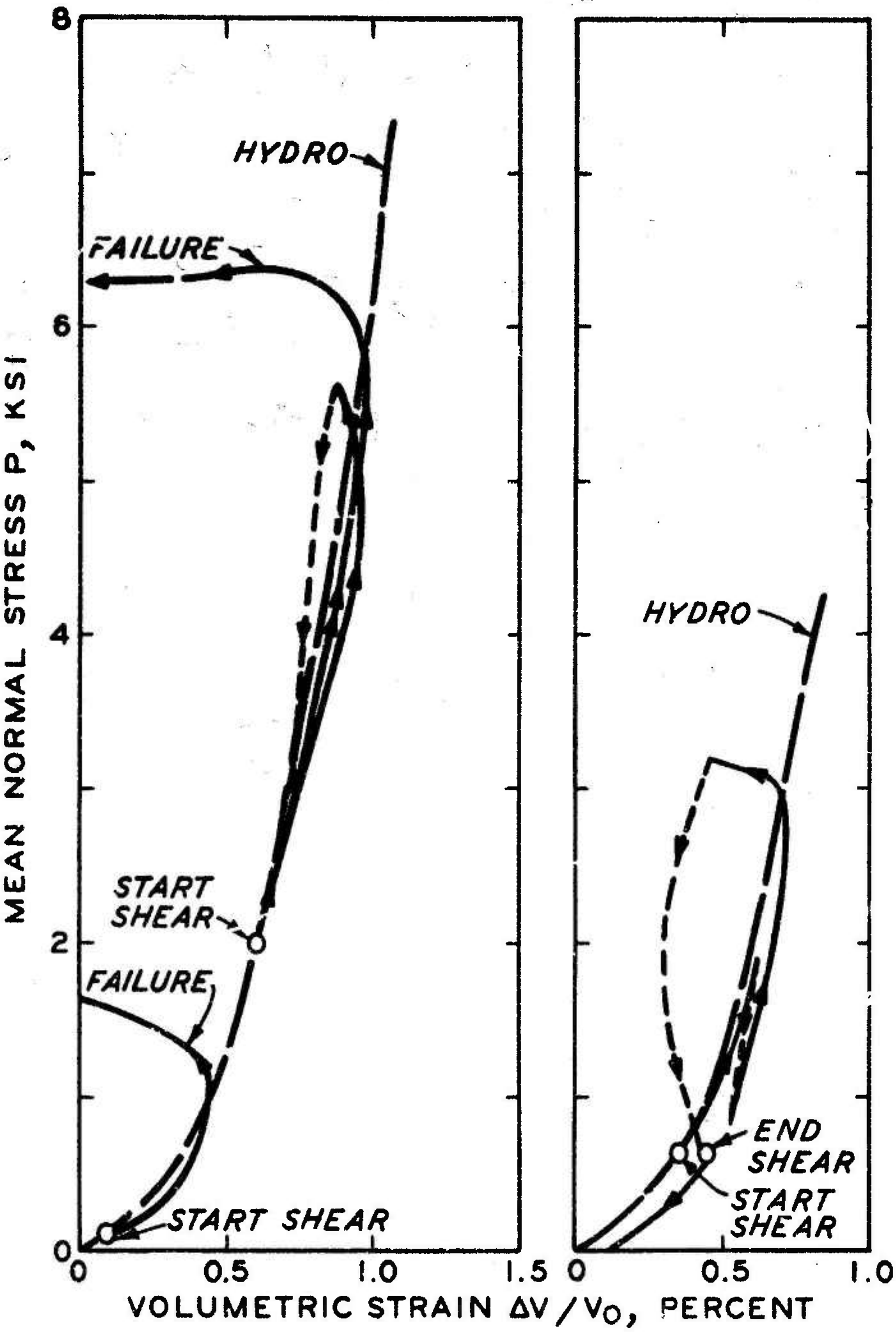


Figure 3.9 Mean normal stress versus volumetric strain showing the typical response of sandstones during TX shear.

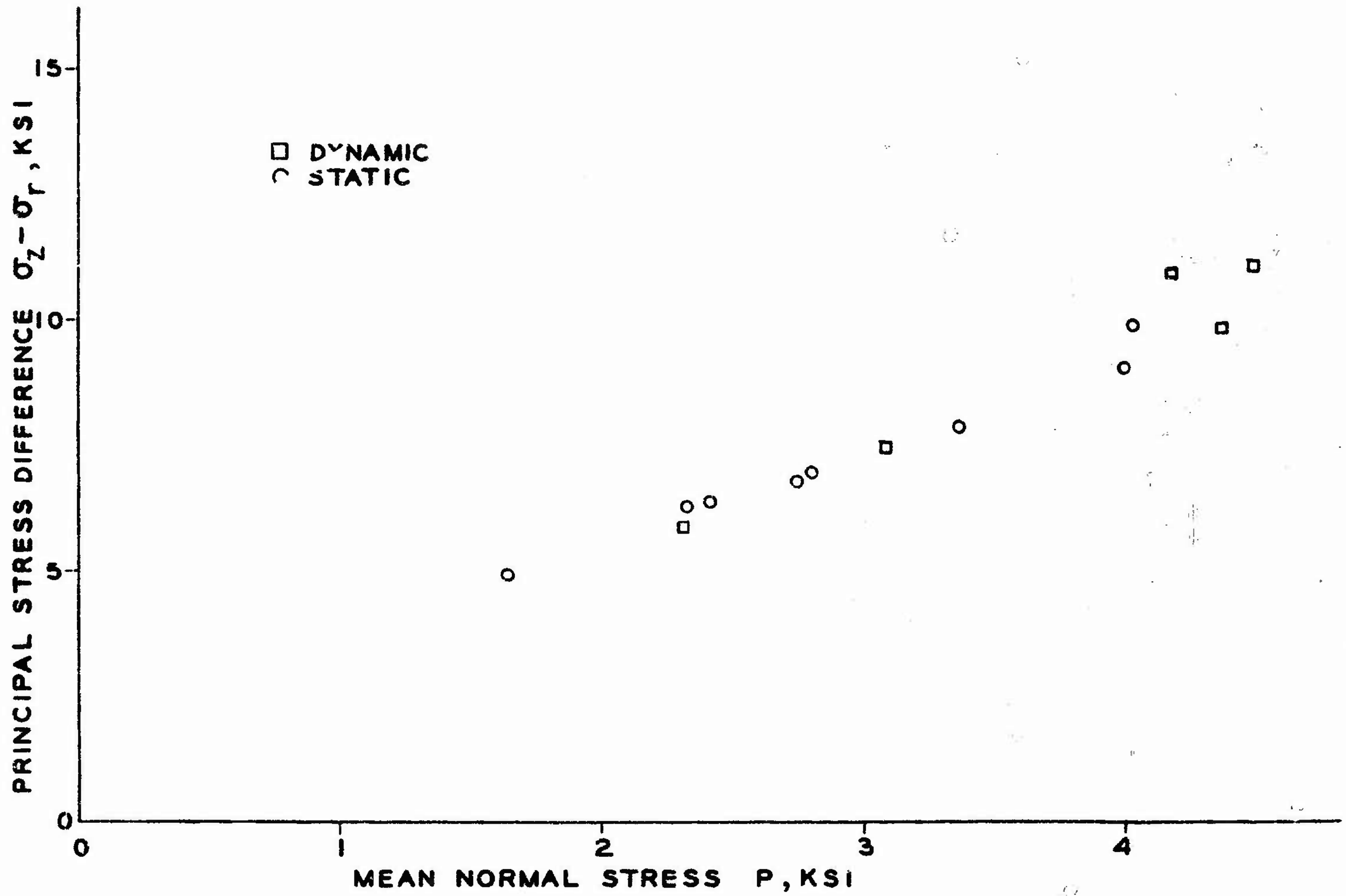


Figure 3.10 Typical static and dynamic TX failure data for sandstones from shallow (<25 feet) depths.

CHAPTER 4

ANALYSIS

The final task of Project LN 310 was the analysis of the data to develop the computational site profile and corresponding constitutive properties. The analysis involved synthesizing all the available information obtained from the literature, the field investigation, and the laboratory tests into an idealized layered profile and matching set of constitutive properties which hopefully would describe the response of the in situ materials to an imposed dynamic loading. Obviously, in many cases the laboratory data are insufficient and/or not representative of field conditions and assumptions must be made. Judgments must be imposed on data scatter and data trends since numerical averaging does not account for the relative quality of the various items of data. A summary of the procedures or approach used in the analysis is presented in this chapter. The general conclusions that were reached in the assumptions required to fill in the gaps in these findings are also listed herein. The idealized geologic profile and typical examples of the representative properties which were recommended for preshot prediction calculations are shown.

4.1 APPROACH

The computational site profile with associated constitutive properties of the Mixed Company Event III site was developed after analyses of all the available information, including the site geology, seismic survey results, boring logs, and composition property data summarized in Chapter 2 and the laboratory stress-strain and strength data discussed in Chapter 3. Where no data existed, such as for the deeper sandstone formations, use was made of literature and unpublished data for similar materials. The information for the various materials encountered was grouped as to material type and/or physical composition properties. The data for each grouping were examined and supplemented where necessary by judgment and/or data from the literature. General conclusions were then reached as to basic data trends and necessary

assumptions made to relate these trends to qualitative behavior patterns for the recommended in situ constitutive properties. A horizontally layered idealized profile was established and specific values selected for key composition properties, such as density, saturation, and air void content. Finally, a set of constitutive properties consisting of a UX stress-strain relation, a stress path for the UX condition, and a TX failure envelope was quantitatively defined for each layer of the recommended calculational profile.

4.2 GENERAL CONCLUSIONS AND ASSUMPTIONS

Two tables are presented which briefly summarize general conclusions and assumptions; Table 4.1 pertains to development of the profile and Table 4.2 to the associated constitutive properties. The statements listed in the two tables are presented solely to indicate some of the gyrations required in order to transform the results of the Mixed Company field and laboratory investigations into a useable format for developing the mathematical constitutive model inputs to ground shock calcuations. Generalization and oversimplification in the statement of many conclusions and assumptions were dictated by the requirement of brevity. The reader is cautioned that these conclusions and assumptions do not apply to any other site; in fact, after analysis of data generated subsequent to June 1972, they may have to be revised substantially even for the Mixed Company III site.

4.3 SPECIFIC RECOMMENDATIONS

The site profile and corresponding compositional properties recommended for use in preshot ground shock predictions of Mixed Company Event III are presented in Table 4.3. Properties for the first layer, i.e., the soil overburden, were initially developed based on an assumed 117-pcf wet density for the fill to be constructed along the main gage line. Those properties were amended upon receipt of early construction data which indicated a substantially lower wet density (111 pcf) for this material.

The UX relations for the layers representing the Kayenta materials

(i.e., Layers II through V) were defined to a stress level of 30 kbar; the TX failure envelopes were given to 20 kbar. Portions of the UX stress-strain loading-unloading relations recommended for Layers I through V are shown in Figure 4.1. Figure 4.2 shows representative stress paths for the state of uniaxial strain along with TX failure envelopes for these same five layers. The complete set of properties was furnished to DNA[1] for use in developing model fits by Weidlinger Associates under Project LN 312 and other interested calculators.

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[1] J. G. Jackson, Jr., letters transmitting Mixed Company site profiles and material property recommendations, to Mr. C. B. McFarland, Defense Nuclear Agency, dated 30 May, 23 June, 29 June, and 10 August 1972, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

TABLE 4.1 CONCLUSIONS AND ASSUMPTIONS USED TO DEVELOP REPRESENTATIVE PROFILE

General Conclusions	Assumptions
1. A berm was placed over the soil at a density approximately equal to the in situ density of the soil at a depth of 2 feet.	1. The site was idealized as a horizontally layered profile, uniform in all directions.
2. The natural soil overburden increased in density (95 to 120 pcf) and degree of cementation down to an indistinct interface with the underlying weathered siltstone of the Kayenta Formation at a depth of 1 to 8 feet.	2. The material comprising each layer was homogeneous and isotropic in all physical properties.
3. The weathered siltstone encompassed an approximately 4-foot-thick zone; it increased in density with depth from 130 to 140 pcf.	3. The fill over the natural soil resulted in a uniform 5-foot thickness of overburden.
4. The remaining portion of the Kayenta Formation could be described as a combination of sandstones, siltstones, mudstones, and conglomerates; no obvious layering pattern, either horizontally or vertically, was discernible. This material had a wide density range (145 ± 10 pcf).	4. The Kayenta Formation could be subdivided into four layers. The uppermost of these was the weathered siltstone. The low-strength and low-moduli clayey conglomerate, which occurred randomly throughout the formation, was assumed to be lumped into a single stratum which formed the third idealized Kayenta layer. The second and fourth layers were based on the properties of more competent specimens from the upper one-third and the lower two-thirds of the formation, respectively.
5. The uniform crossbedded Wingate sandstone occurs at a depth of about 70 feet. The dry density of this material is approximately 120 pcf and is uniform to a depth of at least 150 ft (i.e., the depth of the deepest boring at the site).	5. The Wingate and Chinle Formations and Precambrian basement rock each comprised a layer.
6. The geologic investigation indicated that the Wingate Formation extends from a depth of 70 ± 20 to 400 ± 25 feet.	6. The seismic velocity of the weathered siltstone portion of the Kayenta was 2,500 fps. The seismic velocity of the three underlying layers in this formation was assumed to be approximately 7,500 fps.
7. Below the Wingate Formation is the Chinle Formation, which extends from a depth of 400 ± 25 to 500 ± 50 feet. Precambrian basement rocks underlie the Chinle Formation.	7. The seismic velocity of the Wingate Formation was assumed to be no greater than that of the deeper part of the Kayenta (i.e., 7,500 fps). The velocity of the Chinle Formation was assumed to be 9,000 fps, i.e., somewhat higher than those of the Kayenta and Wingate Formations.
8. The field refraction seismic survey indicated a two-layer profile: 2,000- to 3,000-fps velocities to a depth of about 9 feet and 6,000- to 8,800-fps velocities below 9 feet.	8. A 15,000-fps seismic velocity was assumed for the Precambrian rocks.
9. The weathered siltstone had a rock quality index (RQD) of 0 percent. Unweathered Kayenta materials had RQD's from 90 to 100 percent.	9. None of the formations in the profile were assumed to be saturated.

TABLE 4.2 CONCLUSIONS AND ASSUMPTIONS USED TO DEVELOP REPRESENTATIVE CONSTITUTIVE PROPERTIES

General Conclusions

1. Only the data available by June 1972 would be used to develop representative properties for the preshot calculations.

2. All tests were performed on vertically oriented specimens. Most tests were conducted on specimens that were large enough to include some nonuniformities such as clay seams and/or natural fractures.

3. The laboratory tests were conducted with times to peak stress ranging from several microseconds to several seconds.

4. Most specimens that were tested at peak stress levels less than 1 kbar had their in situ water contents preserved. Most specimens tested at higher pressures were tested in an air-dried condition.

5. The soil UX and TX response was dominated by the influence of density and water content variations. Some effects due to loading rate were also observed.

6. The weathered siltstones and unweathered clayey conglomerates of the Kayenta Formation were considerably more compressible and somewhat weaker than the unweathered Kayenta sandstones, siltstones, and mudstones.

7. For specimens tested with loading times ranging from seconds to milliseconds, the UX curves were initially soft but stiffened gradually under stresses up to 1,000 psi; thereafter, a marked stiffening was noted. Rate effects were indistinguishable due to the dominance of other variables. UX gun test data with microsecond loading rates indicated much stiffer initial loading moduli, i.e., values comparable to those observed at the highest stress levels in the slower tests.

8. For TX specimens tested with loading times ranging from seconds to milliseconds, the influences due to rate of loading were also indistinguishable due to scatter resulting from other more dominant variables.

9. Horizontal tensile strengths, as indicated by Brazil test results, were up to five times the vertical tensile strengths indicated by results of direct-pull tests. Although the quantitative validity of the Brazil test results is questionable, they definitely indicate that horizontal tensile strengths equal or exceed vertical strengths.

Assumptions

1. The times to peak stress anywhere within the site due to the expected detonation would be on the order of several milliseconds.

2. All materials were assumed to be isotropic, in spite of conflicting tensile test evidence. All recommended properties, including those for tension, were based on vertically oriented test specimens.

3. The density and water content of the constructed fill or berm should be used as the basis for selecting representative properties for Layer I.

4. At low stress levels ($\ll$10 psi), the initial UX moduli would be equal to moduli calculated from field seismic velocities. At pressures above the level of structural collapse, the UX stress paths of the Kayenta materials would be similar to those observed for sand.

5. Many of the open fractures found in samples from the weathered Kayenta siltstone (Layer II) were due to sampling disturbances; therefore, initial strains in excess of 2 percent resulting under UX tests loadings less than 10 psi were ignored.

6. A major proportion of the in situ mass of weathered Kayenta siltstones (Layer II) contained numerous healed cracks, laminations, and fibrous root systems; therefore, recommended compression properties were weighed in favor of UX test results from specimens containing such features.

7. Since insufficient TX data were available for the weathered siltstones (Layer II) to define shear behavior, strength data for unweathered materials could be reduced arbitrarily to define weathered strengths.

8. For the unweathered materials, since rock quality index values were always greater than 90 percent, responses observed for laboratory tests on specimen cores adequately depicted in situ mass behavior.

9. Laboratory test results for sandstone, siltstone, and mudstone specimens obtained from the upper one-third of the unweathered Kayenta Formation adequately depicted the in situ mass behavior of Layer III.

10. Laboratory test results for clayey conglomerate specimens obtained from throughout the unweathered Kayenta Formation adequately depicted the in situ mass behavior of the extremely idealized Layer IV.

11. Laboratory test results for sandstone, siltstone, and mudstone specimens obtained from the lower two-thirds of the unweathered Kayenta Formation adequately depicted the in situ mass behavior of Layer V.

12. Layer VI represented the Wingate Formation and was slightly softer in compression and slightly weaker in shear than Layer V because of its higher porosity and markedly lower density.

13. Layers VII and VIII represented the Chinle Formation and Precambrian basement rock, respectively, and behaved as linearly elastic materials; properties were based on data found in the literature for other rocks of the same type and age.

TABLE 4.3 REPRESENTATIVE PROFILE AND COMPOSITIONAL PROPERTIES

Depth	Layer	Wet Density	Dry Density	Saturation	Volume of Air	Seismic Compressior Velocity	Material Type
feet		pcf	pcf	percent	percent	fps	
0 to 5	I	111.3	104.0	31.0	25.9	1,800	Soil overburden
>5 to 9	II	137.0	132.4	36.7	12.8	2,500	Weathered Kayenta siltstone
>9 to 18	III	147.0	140.7	66.5	5.1	7,100	Upper Kayenta materials
>18 to 22	IV	154.0	148.1	80.6	2.3	7,500	Soft Kayenta material
>22 to 70	V	147.0	140.7	66.5	5.1	7,500	Lower Kayenta material
>70 to 400	VI	128.4	120.0	49.1	14.0	7,400	Wingate Formation
>400 to 500	VII	158.0	--	--	--	9,000	Chinle Formation
>500	VIII	162.0	--	--	--	15,000	Precambrian basement rock

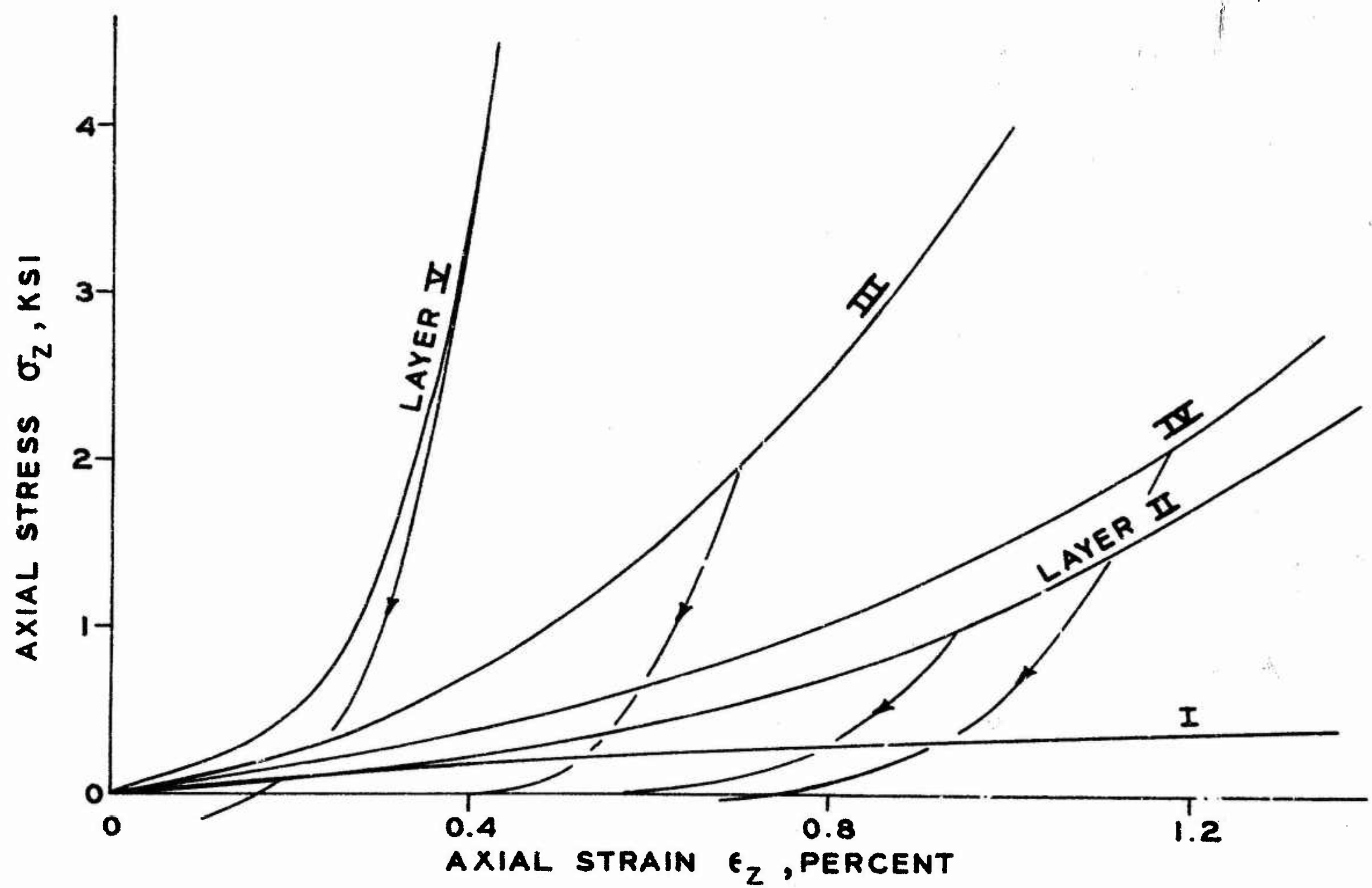

Figure 4.1 Representative UX stress-strain loading and unloading relations for Layers I through V.

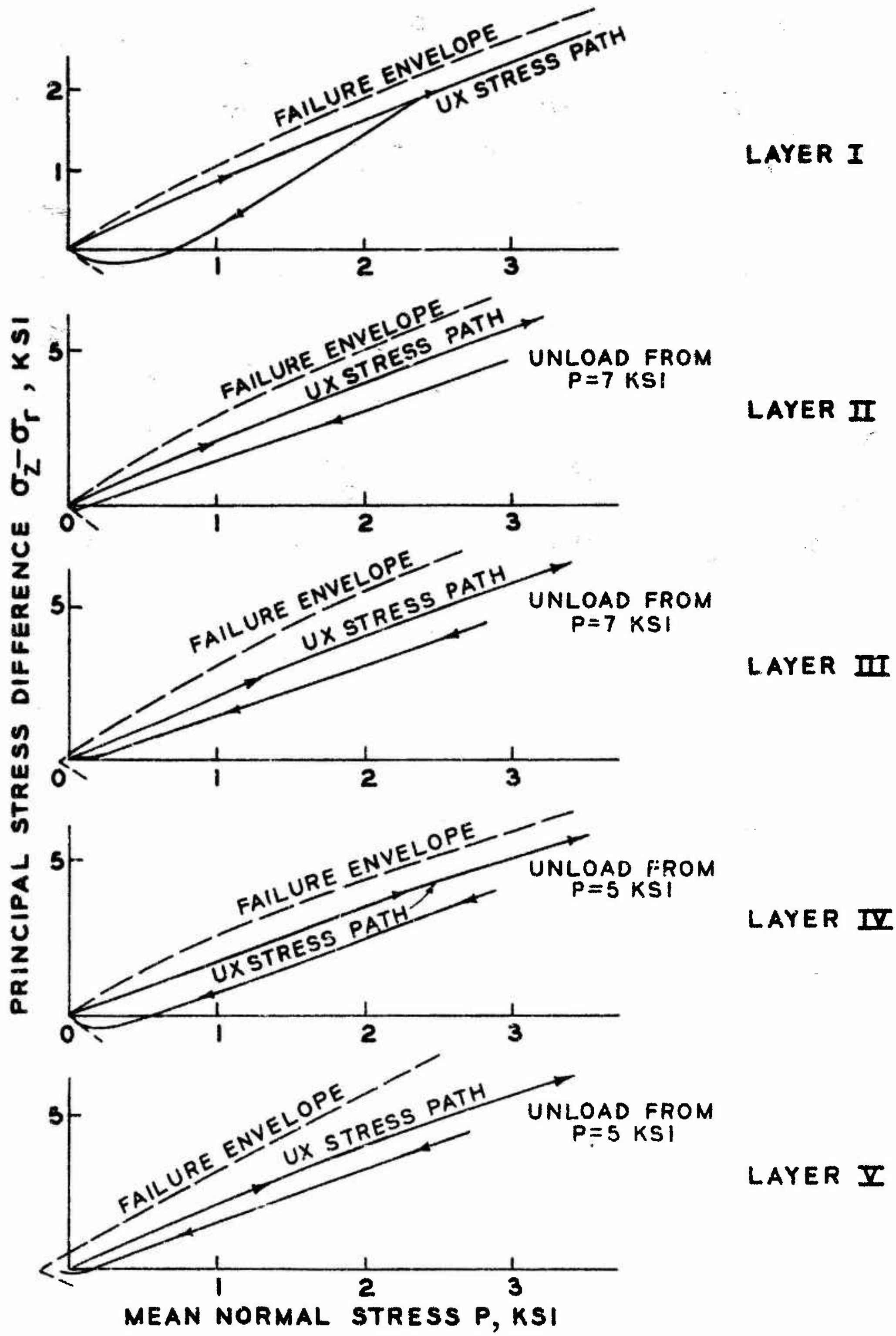

Figure 4.2 Representative UX stress paths and TX failure envelopes for Layers I through V.

CHAPTER 5

EPILOG

The purpose of this report was to summarize the preshot field and laboratory work performed under Project LN 310, "Soil Sampling and Laboratory Testing for Constitutive Relations." The site geology as well as the field and laboratory test programs carried out have been described. Typical test results and the representative constitutive properties furnished for use in preshot calculations have been presented. While it was necessary, for the purposes of calculations, to idealize the near-surface sedimentary rocks of the Kayenta Formation into a few homogeneous layers, the data indicate considerable variation in lithology and constitutive properties within this formation. Thus, while the siting objective of obtaining a shallow soil layer was achieved, the data indicate that in a strict sense the objective of obtaining a nearly uniform underlying sandstone stratum was not achieved.

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below:

Ehrgott, John Q
Preshot material property investigation for the Mixed Company site: summary of subsurface exploration and laboratory test results, by J. Q. Ehrgott. Vicksburg, Miss., U. S. Army Engineer Waterways Experiment Station, 1973.
66p. illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper S-73-62)
Sponsored by Defense Nuclear Agency, Subtask SB209, Work Unit 11, Laboratory studies of the response of soil and rock to blast-type loadings.
1. Constitutive properties. 2. Earth materials. 3. Field investigations. 4. Laboratory tests. 5. Mixed Company (Event). 6. Subsurface exploration. I. Defense Nuclear Agency. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper S-73-62)
TA7.W34m no.S-73-62